भारतीय मानक
Indian Standard

IS 15026 (Part 1) : 2022

चट्टानों में सुरंग बनाना — दिशानिर्देश

भाग 1 सुरंग बनाने की पारंपरिक पद्धति

(पहला पुनरीक्षण)

Tunnelling in Rock Masses — Guidelines

Part 1 Conventional Tunnelling Methods

(First Revision)

ICS 93.060

© BIS 2022

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS
मानक भवन, 9 बहादुरशाह ज़फर मार्ग, नई दिल्ली – 110002
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI-110002
www.bis.gov.in www.standardsbis.in

November 2022 **Price Group 12**

Rock Mechanics Sectional Committee, CED 48

FOREWORD

This Indian Standard (Part 1) (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Rock Mechanics Sectional Committee had been approved by the Civil Engineering Division Council.

Tunnelling is an art practised by all engineers, geologists, planners and people. Failures should be regarded as challenges and opportunities for generating new vital knowledge and thereby increasing self-reliance in the tunnelling. The key to success is team spirit and love for rocks and nature.

High overburden poses great challenges to tunnel construction with squeezing ground in weak rock masses and rock bursting in strong rock conditions. Further the shear zones are met frequently in Himalayan region. Special treatment is necessary to support shear zones in the tunnels.

The design of underground excavations is, to a large extent, the design of underground support system according to excavation induced ground behaviour. These can range from no support in the case of a temporary excavation in good rock mass to the use of fully grouted and tensioned bolts or cables with wire mesh; or sprayed fibre reinforced shotcrete (FRS) in average rock mass to the use of reinforced ribs of shotcrete in poor rock mass. Steel rib with backfill concrete may also be required for the support of a large permanent civil engineering excavation.

The philosophy of design of any underground excavation should be to utilize the rock mass itself as the principal structural materials, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of shotcrete or steel supports. In weak zones with water ingress problems, pre-excavation grouting not only controls water ingress but also improves rock mass properties. The extent to which aim of this philosophy can be met depends upon the geological conditions which exist at site and the extent to which the designer and construction engineer is aware of these conditions.

Experience and engineering judgement helps to make a considered and informed decision, but continuous measurements during construction are also essential to compare actual behaviour with those predicted.

There are many difficult geological conditions and extraordinary geological occurrences, such as intra-thrust zones, very wide shear zones, geothermal zones of high temperature, cold/hot water springs, water charged rock masses, intrusions, etc. These are very difficult to forecast. Investigation methods like geophysical tests, probe drilling from drilling jumbo, core drilling, etc should be carried out to detect the extent of their presence for selection of right tunnelling method, treatment and excavation support measures for which experts may be consulted.

This standard was first formulated in 2002. In this revision, to incorporate the latest advancement in tunnelling, the tunnelling method in rock masses has been grouped into two parts. This standard (Part 1) covers guidelines for conventional tunnelling methods. The other part in the series is:

Part 2 Mechanized tunnelling methods (*under preparation*)

This revision incorporates the following major modifications:

a) Elaborate details of the advance probing technique which include probe holes, tunnel geophysical probing and advance core drilling have been added;
b) Improved details of classification of ground behaviour have been added;
c) Separate clause has been added for shotcrete with specifications of materials for wet mix shotcrete;
d) Provision of polypropylene fibres has been added;
e) Provision of tunnel bolts like water expandable friction bolts, grouted rock bolts, self-drilling bolts and cable bolts have been included; and
f) Provision of tunnel supports like lattice girders and reinforced ribs of shotcrete have been included.

The composition of the Committee responsible for the formulation of this standard is given in Annex A.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

TUNNELLING IN ROCK MASSES — GUIDELINES

PART 1 CONVENTIONAL TUNNELLING METHODS

(First Revision)

1 SCOPE

This standard (Part 1) provides guidelines for construction of tunnels using conventional tunnelling methods.

2 REFERENCES

The standards given below contain provisions which through reference in this text, constitute provision of this standard. At the time of publication, the editions indicated were valid. Any standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
269 : 2015	Ordinary Portland cement — Specification (*sixth revision*)
432	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement:
(Part 1) : 1982	Mild steel and medium tensile steel bars (*third revision*)
(Part 2) : 1982	Hard-drawn steel wire (*third revision*)
456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
800 : 2007	General construction in steel — Code of practice (*third revision*)
4880 (Part 6) : 1971	Code of practice for design of tunnels conveying water: Part 6 Tunnel supports
5878	Code of practice for construction of tunnels conveying water:
(Part 4) : 1971	Tunnel supports
(Part 6) : 1975	Steel lining
9012 : 1978	Recommended practice for shotcreting
9103 : 1999	Concrete admixtures — Specification (*third revision*)
13365 (Part 2) : 2019	Quantitative classification systems of rock mass — Guidelines: Part 2 Rock mass quality for prediction of support pressure, support system and engineering properties in underground openings (*first revision*)
15388 : 2003	Silica fume — Specification
16481 : 2022	Textiles — Synthetic micro fibres for use in cement based matrix — Specification (*second revision*)
16993 : 2018	Microfine ordinary portland cement — Specification
17446 : 2020	Observational method for tunnelling in rock masses — Guidelines

3 ADVANCE PROBING

Long tunnels, sometime pass through complex geological conditions particularly in case of deep tunnels. It is extremely difficult to anticipate geological conditions in deep seated tunnels merely based on surface observations.

3.1 Probe Holes

Each round of probing inside tunnels should consist of at least 3 probe holes near tunnel periphery along the tunnel alignment with outlooking angle and located depending upon nature of anticipated problem or joint pattern of rock mass after discussion with the geologists. If the rock mass condition permit, each round of probe holes should be drilled preferably up to 18 m to 20 m length along the tunnel alignment with an overlap of atleast 5 m between successive rounds. Modern day drilling jumbos are equipped with sensors which record various measure while drilling (MWD) parameters, namely penetration rate, feed pressure, rotation pressure, rotation speed, flushing flow, flushing pressure, etc along the depth of each probe hole drilled by the machine. The data from the machine can be visualized and graphically interpreted through drilling jumbo software. These probe holes give a fair idea about rock mass condition ahead of the face. It helps in suggesting the strategy of tunnelling.

3.2 Tunnel Geophysical Probing

Tunnel seismic prediction is a popular method to probe and predict complex geological conditions well in advance. The tunnel seismic prediction method detects changes in rock mass, such as irregular bodies, discontinuities, faults and fracture zones ahead of the tunnel face. Acoustic signals comprising compression (P) and shear (S) wave, are produced by a series of small blast shots of usually 50 g to 200 g of detonation cord aligned along one tunnel wall side and the reflected waves are captured through three component receivers fixed in the tunnel wall. With help of software, data can be processed and evaluated to determine rock mechanical properties, such as poisson's ratio and static and dynamic young's moduli within the prediction area. Inadequate contrast in physical properties of rock mass and less than 25° dip or strike angles compared to tunnel axis in the prediction area limits the effective usage of this test. Usually, depth of prediction obtained by this test method is in the range of 100 m to 125 m depending on the properties of rock mass conditions. The test should be performed as a risk control tool for determining exact location of fault zones, cavities or water bearing formations as predicted in the tunnel geological model.

Other geophysical techniques may also be used for ascertaining rock conditions ahead of tunnel face.

3.3 Advance Core Drilling

In critical cases and where special investigations are required, advance core drilling can also be performed to ascertain various geotechnical parameters and complex geological conditions.

4 EFFECT OF SEISMICITY

A tunnel in a seismic area is likely to be affected near the portals and in neighbourhood of faults and thrusts. The effect is observed to be up to a distance along the tunnel within $\pm B$ on both sides of the faults/thrusts, where B is span/size of the opening. The design support pressure in the affected length of tunnel may be taken as 1.25 times the ultimate support pressure [*see* IS 13365 (Part 2)].

5 TUNNEL INSTRUMENTATION AND MONITORING

5.1 Instrumentation of tunnel openings should be done where squeezing ground condition is expected. The survival rate of tunnel instruments is generally as low as 30 percent. Therefore many sections of the tunnel should be instrumented so that enough instruments survive and reliable data is obtained. The post-monitoring of support system in squeezing ground should also be carried out until support system has stabilized with time. In cases of squeezing ground conditions, observed vertical and horizontal tunnel closures should be controlled to less than four percent of tunnel width and height, respectively.

5.2 Monitoring and instrumentation strategy should be to verify design assumptions, assess stability and timely adjust tunnelling strategy, improve ground modelling, evidence collection and back analysis to have greater understanding of rock mass behaviour (*see* IS 17446).

5.3 Monitoring and instrumentation programme should define expected behaviours and acceptable limits, instrumentation layout and frequency for observations, identification of roles and responsibilities for quick analysis of results, safety management plan with contingencies.

6 SELECTION OF TYPE OF SUPPORT SYSTEM

6.1 Table 1 classifies various possible ground behaviours for tunnelling. Rock bursts are rare in tunnelling for civil engineering projects. Table 2 suggests the method of excavation, type of supports and precautions for various ground behaviours.

Squeezing is ruled out if observed *in-situ* maximum tangential strain (ratio of downward crown displacement to the tunnel radius) is less than the critical strain [ratio of uniaxial compressive strength (UCS) to modulus of elasticity of rock material] in case of properly supported tunnels.

6.1.1 Before taking up the design of supports, the rock load and pressure likely to act on the supports shall be estimated. The determination of rock load is complex problem. This complexity is due to inherent difficulty of predicting the primary stress conditions in the rock mass (prior to excavation) and also due to the fact that the magnitude of the secondary pressure developing after the excavation of the cavity depends upon a large number of variables, such as size and shape of cavity, depth of cover, disposition of strike and dip of rock formation in relation to alignment of tunnel, method of excavation, period of time elapsing between excavation and the time when the rock is supported and the rigidity of support. These pressures may not develop immediately after excavation but may take a long period after excavation to develop due to adjustment/displacement in the rock mass.

6.1.2 In major tunnels it is recommended that as excavation proceeds, load cell measurements and diametrical change measurements be carried out so that rock loads may be correctly estimated. In rock where the loads and deformation do not attain stable values, it is recommended that pressure measurements should be made by using flat jack or pressure cells.

Table 1 Classification of Ground Behaviours for Tunnelling

(*Clause* 6.1)

Ground Behaviour Type				**Ground Behaviour**	**Remarks**
Gravity Driven	**Sudden**	1 Stable		The surrounding ground will stand unsupported for several days or longer.	Massive, durable rocks at low and moderate depths.
		2 Block fall(s)	of single blocks	Stable, with the potential fall of individual blocks.	Discontinuity-controlled failure.
			of several blocks	Stable with potential fall of several blocks (slide volume < 10 m^3).	
		3 Cave-in		Inward, quick movement of larger volumes (> 10 m^3) of rock fragments or pieces.	Encountered in highly jointed or crushed rock
		4 Running ground		A particular material quickly invades the tunnel until a stable slope is formed at the face. Stand-up is time zero or nearly zero.	Examples are clean medium to coarse sands and gravels above groundwater level.
Stress Induced	**Time dependent**	5 Buckling		Breaking out of fragments in tunnel surface.	Occurs in anisotropic hard, brittle rock under sufficiently high load due to deflection of the rock structure.
		6 Rupturing from stresses		Gradually breaking up into pieces, flakes or fragments in the tunnel surface.	The time-dependent effect of slabbing or rock burst from redistribution of stresses.
		7 Slabbing		Sudden, violent detachment of thin rock slabs from sides or roof.	Moderate to high overstressing of massive hard brittle rock, includes popping or spalling.
		8 Rock burst		Much more violent than slabbing and involves considerably larger volumes.	Very high overstressing of massive hard, brittle rock of considerably larger volumes (heavy rock bursting often registers as a seismic event).
		9 Squeezing		Time-dependent deformation, essentially associated with creep caused by overstressing. Deformations may terminate during construction or continue over a long period.	Overstressed plastic, massive rocks and materials with a high percentage of micaceous minerals or of clay minerals with a low swelling capacity. Mild squeezing (u_a/a = 1.3 percent). Moderate squeezing (u_a/a = 3.5 percent). High squeezing (u_a/a > 5 percent).
Water Influenced	**Time dependent**	10 Ravelling from slaking		Ground breaks gradually up into pieces, flakes or fragments.	Disintegration (slaking) of some moderately coherent and friable materials. Examples: mudstones, stiff and fissured clays.
		11 Swelling	of certain rocks	Advance of surrounding ground into the tunnel due to expansion caused by water adsorption. The process may sometimes be mistaken for squeezing.	Occurs in swelling of rocks, in which anhydrite, halite (rock salt) and swelling clay minerals, such as smectite (montmorillonite), constitute a significant portion.
			of certain clay seams or fillings	Swelling of clay seams caused by adsorption of water. This leads to loosening of blocks and reduced shear strength of clay.	The swelling takes place in seams having fillings of swelling clay minerals (smectite, montmorillonite).
		12 Water ingress		Pressurized water invades the excavation through channels or openings in rocks.	May occur in porous and soluble rocks, or along significant openings or channels in fractures or joints.
	Sudden	13 Flowing ground		A mixture of water and solids quickly invades the tunnel from all sides, including the invert.	May occur in tunnels below the groundwater table in particulate materials with little or no coherence.

u_a = radial tunnel closure; α = tunnel radius; and

u_a/a = normalized tunnel closure in percentage.

Table 2 Method of Excavation, Type of Supports and Precautions to be Adopted for Different Ground Behaviours

(*Clauses* 6.1 *and* 12.5.1)

Sl No.	Ground Behaviour Type	Excavation Method	Type of Support	Precautions
i)	Block falls	Full-face drill and controlled blast with localized light forepoles	Flexible support: Shotcrete and systematic bolting (end anchored and pre-tensioned/friction bolts); fibre reinforced shotcrete (FRS) may or may not be required	Sealing layer of shotcrete should be applied immediately after scaling. Subsequent layer of shotcrete should be applied after some delay but within the stand-up time to release the strain energy of rock mass
ii)	Cave-in or running and flowing	Multiple drift method with heavy fore-poles; sometimes advance grouting of the ground is essential; shield tunnelling may be used in soil like condition	Full column grouted rock anchors and FRS, face sealing shotcrete, face bolting with glass fibre bolts, steel supports depending upon overburden and in exceptional cases with shield tunnelling	Progress is very slow. Trained crew should be deployed. Groutability tests of soil like strata may be required to decide type of grout material.
iii)	Buckling or rupturing from stresses or slabbing	Full face drill and blast	Fibre reinforced shotcrete with full column grouted rock anchors/resin anchors immediately after excavation	Use forepoling if stand-up time is low
iv)	Rock burst	Full face drill and blast	Fibre reinforced shotcrete with full column grouted ductile rock anchors/resin anchors immediately after excavation.	Micro-seismic monitoring is essential
v)	Mild squeezing	Heading and bench, drill and blast	Full column grouted rock anchors and FRS, floor to be shotcreted to complete supporting. Lattice girders/reinforced ribs of shotcrete may or may not be required.	Install support after each blast; circular shape is ideal; side pressure is expected; do not have a long heading which delays completion of support ring.
vi)	Moderate squeezing	Heading and bench, drill and blast	Flexible support full column grouted ductile rock anchors and FRS. Lining stress controllers with lattice girders. Floor bolting to avoid floor heaving, to develop a reinforced rock frame; sliding steel ribs, if required, should be installed and embedded in shotcrete for immediate activation	Install support after each blast; increase the tunnel diameter to absorb desirable closure; circular shape is ideal; side pressure is expected; instrumentation is essential; ductile elements like rock bolts and steel supports should be compatible in load sharing
vii)	High squeezing	Full face in small tunnels and heading and bench in large tunnels; use forepoling if stand-up time is low	Very flexible support, full column grouted ductile rock anchors and FRS; atleast four rows of lining stress controllers spread along invert and overt with lattice girders/yielding steel ribs with struts embedded in shotcrete; for immediate activation, close ring by erecting invert support; floor bolting to avoid floor heaving; sometimes steel ribs with loose backfill is also used to release the strain energy in controlled manner (tunnel closure more than 4 percent shall not be permitted)	Increase the tunnel diameter to absorb desirable closure; provide invert support as early as possible' to mobilise full support capacity long term instrumentation is essential; circular shape is ideal; invert strut may be required to obtain ring closure in heading; face bolting with glass fibre bolts/self-drilling anchors may be required to control face extrusion
vii)	Ravelling	Heading and bench; drill and blast	Steel support with struts/pretensioned rock bolts with fibre reinforced shotcrete may or may not be required	Expect heavy loads including side pressure
ix)	Swelling	Full face or heading bench: drill and blast	Full column grouted rock anchors with SFRS shall be used around the tunnel; increase 30 percent thickness of shotcrete due to weak bond of the shotcrete with rock mass; erect invert strut; the first layer of shotcrete is sprayed immediately to prevent ingress of moisture into rock mass	Increase the tunnel diameter to absorb the expected closure; prevent exposure of swelling minerals to moisture monitor tunnel closure

6.1.3 In the absence of any data of instrumentation, rock load or support pressure may be estimated by *Q* system as given in IS 13365 (Part 2).

6.1.4 As the tunnels generally pass through different types of rock formations, it may be necessary to workout alternative cross-sections of the tunnel depicting other acceptable types of support systems. These types may be selected to match the various methods of attack that may have to be employed to get through the various kinds of rock formations likely to be encountered. The 'A' and 'B' lines shall be shown on these sections.

6.1.5 The support system shall be strong enough to carry the ultimate loads. For a reinforced concrete lining it is economical to consider the steel supports as an integral part of the permanent lining.

6.1.6 Temporary support system shall be installed within stand-up time for safety of workmen but not too early.

7 SHOTCRETE

7.1 General

7.1.1 Shotcrete can be unreinforced, reinforced with anchored mesh or with steel/polypropylene fibres in the mix. All loose rock shall be scaled out and washed before applying shotcrete. Shotcrete for tunnel supports may be used by itself as a thin skin type reinforcement or used in combinations with rock bolts, steel supports and other more conventional tunnel reinforcements.

Before shotcreting all loose rocks are sealed and washed before applying shotcrete. Shotcrete is forced into open joints, fissures, seams and irregularities in the rock surface and in this way it serves the same binding function as mortar in a stone wall. Shotcrete hinders water seepage from joints and seams in the rock and thereby prevents piping of joint tilling materials and air and water deterioration of the rock. Shotcrete's adhesion to the rock surface and its own shear strength provide a considerable resistance to the fall of loose rock blocks from the roof of a tunnel. A thicker shotcrete layer (150 mm to 250 mm) provides structural support, either as a closed ring or as an arch type member.

7.2 Mix

Shotcrete shall be applied by either the wet or dry process as appropriate to the circumstances. Compatibility for various proportions of mix shall be verified before finalizing the mix. Shotcrete shall be a mixture of cement, sand and aggregate.

7.2.1 *Cement*

7.2.1.1 Ordinary Portland cement conforming to requirements of IS 269, having higher fineness is considered suitable for shotcrete application.

7.2.1.2 Minimum cement content shall not be less than 380 kg/m^3 and minimum total binder content shall not be less than 400 kg/m^3.

7.2.2 *Aggregates*

7.2.2.1 The gradation curve of aggregates should be normally within the minimum and maximum limit given in Table 3.

Table 3 Recommended Aggregate Gradation Zone

(*Clause* 7.2.2.1)

Sl No.	IS Sieve Size	Percentage Passing	
	mm	Minimum	Maximum
(1)	(2)	(3)	(4)
i)	0.125	4	12
ii)	0.25	11	26
iii)	0.5	22	50
iv)	1.0	37	72
v)	2.0	55	90
vi)	4.0	73	100
vii)	8.0	90	100
viii)	10.0	94	100
ix)	12.5	97	100
x)	16.0	100	100

7.2.3 *Additives*

Densified silica fume conforming to IS 15388 may be added up to 10 percent by weight of cement to improve strength, adhesiveness, cohesiveness, durability, and achieve reduction in rebound without reducing total reactive content in shotcrete mix. Trials shall be carried before ascertaining right dosage.

7.2.4 *Water*

Water shall conform to requirements given in IS 456. For optimum results, water-cement ratio shall be kept in the range of 0.37 to 0.45 for high strength and durability.

7.2.5 *Chemical Admixtures*

7.2.5.1 Chemical admixtures shall conform to requirements given in IS 9103. All admixtures shall be free from chlorides such that percentage of chlorides shall not exceed 0.1 percent by weight.

7.2.5.2 Superplasticizers shall be used to enhance workability of mix and reduce water demand. Preferably, high range water reducing admixtures shall be used which do not have undesirable effect on setting time.

7.2.5.3 Liquid accelerators having alkali content less than 1 percent are preferable. Typical dosage varies in range of 5 percent to 7 percent. Higher dosages of more than 7 percent shall be subject to verification of detrimental effects over long duration of 6 months or more.

7.2.5.4 Typical wet shotcrete mix for M 30 grade is suggested in Table 4. Actual proportions will vary as per actual site conditions and application requirements.

Table 4 Typical Wet Shotcrete Mix

(*Clause* 7.2.5.4)

Sl No. (1)	Material (2)	Quantity in (3)
i)	OPC 53 cement, kg/m^3	380 to 420
ii)	Densified silica fume, kg/m^3	20 to 40
iii)	Fine aggregate, kg/m^3	1 100 to 1 300
iv)	Coarse aggregate, kg/m^3	600 to 800
v)	Steel fibre/poly fibre, kg/m^3	20 to 35/4 to 8
vi)	Superplasticizer, percent	2 to 4
vii)	Liquid accelerator, percent	5 to 7
viii)	Water-cement ratio, kg/m^3	0.37 to 0.45

7.3 Thickness

The thickness of shotcrete required depends upon the type of rock, the extent of stratification and/or joints, blockiness and also the size of the tunnel. The thickness may normally range from 50 mm to 150 mm and whether it should be used plain or fibre reinforced or with wire-mesh anchored to rock will depend upon the actual site conditions in each case. However, the thickness may be more than 150 mm in certain cases based on design and application requirements.

7.4 Support Capacity of Shotcrete in Roof

7.4.1 It is assumed that, shotcrete is intimately in contact with the rock mass and has the tendency to fail by shearing alone. Capacity of shotcrete, P_{sc} is given by:

$$P_{sc} = \frac{2q_{sc}t_{sc}}{F_{sc}B} \quad \dots (1)$$

where

q_{sc} = shear strength of shotcrete (300 t/m^2 in most of the cases);

t_{sc} = thickness of shotcrete (in m);

B = size of opening (in m);

BF_{sc} = horizontal distance between vertical planes of maximum shear stress in shotcrete (in m);

F_{sc} = 0.6 ± 0.05; and

P_{sc} = support capacity of shotcrete lining (in t/m^2).

7.4.2 The thickness of shotcrete may be estimated by substituting ultimate support pressure, P_{roof} for P_{SC} in equation (1). Additional layers of shotcrete should be sprayed to arrest tunnel closure if needed.

7.5 Fibre Reinforced Shotcrete (FRS)

7.5.1 Fibre reinforced shotcrete either alone or in combination with rock bolts (specially in large openings) provides a good and fast solution for both initial and permanent rock support. Being ductile, it can absorb considerable deformation before failure.

7.5.2 Controlled blasting should be used preferably. The advantage of fibre reinforced shotcrete is that smaller thickness of shotcrete is needed, in comparison to that of conventional shotcrete. Fibre reinforced shotcrete is required, especially in rock conditions where support pressure is high. Use of fibre-reinforced shotcrete along with resin anchors is also recommended for controlling rock burst conditions because of high fracture toughness of shotcrete due to especially long fibres. This can also be used effectively in highly squeezing ground conditions. It ensures better bond with rock surface. With mesh, voids and pockets might form behind the mesh thus causing poor bond and formation of water seepage channels as indicated in Fig. 1.

7.5.3 The major draw-back of normal shotcrete is that it is rather weak in tensile, flexural and impact resistance strength. These mechanical properties are improved by the addition of fibres. Deciding a performance criterion (in terms of energy absorption for a fixed deformation value or residual strengths at different deformation values) is mandatory for selecting the type and dosage of fibres for shotcrete application, for which specialist literature may be referred. Most commonly used fibres are steel fibres and polypropylene fibres.

7.5.4 *Steel Fibres*

7.5.4.1 Steel fibres are made in various shapes to increase their bonding intimacy with the shotcrete (*see* Fig. 2). It is found that hooked ends type steel fibres behave more favourably than other types of steel fibres in flexural strength and toughness. Glued fibres should be used to prevent balling of fibres and choking of nozzle at the time of shotcrete application.

7.5.4.2 Steel fibres make up between 0.5 to 2 percent of the total volume of the mix (1.5 to 6 percent by weight). Shotcrete mixes with fibre contents greater than 2 percent are difficult to prepare and shoot.

7.5.4.3 Steel fibres are manufactured by cutting cold drawn wires as per IS 432 (Part 1) and IS 432 (Part 2).

7.5.4.4 Some of the important parameters of steel fibres shall be:

a) Geometrical shape – Typically as shown in Fig. 2. Length of the fibres may be 20 mm to 40 mm. Recommended sizes of the fibres are 25 mm to 35 mm. Equivalent diameter of the steel fibre may be in a range of 0.40 mm to 0.60 mm.

b) Aspect ratio (length/equivalent diameter) = 60 to 75.

c) Ultimate tensile strength > 1 000 MPa

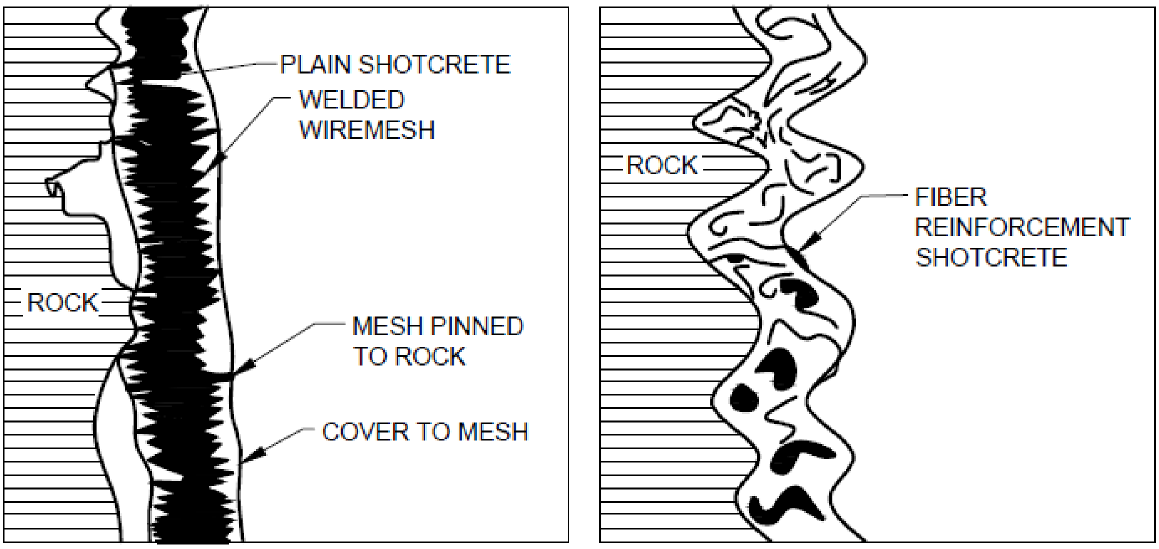
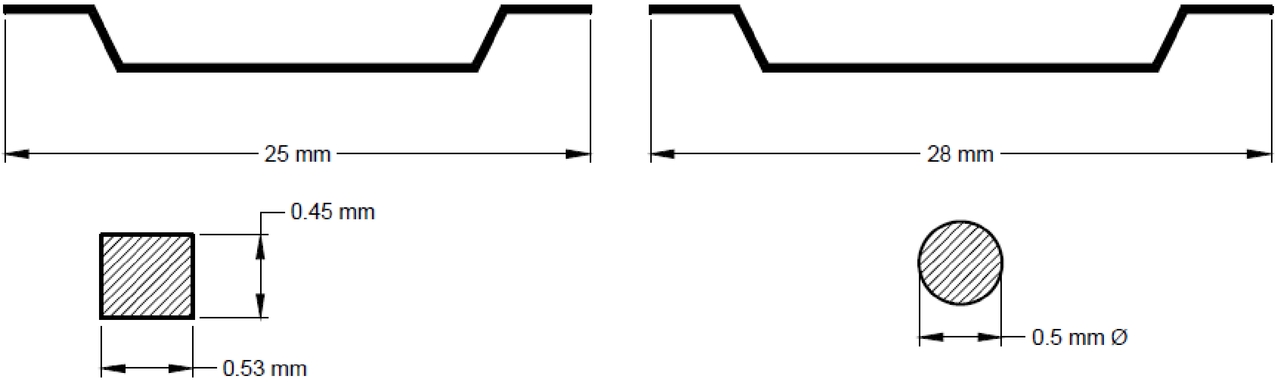


FIG. 1 DIFFERENCE IN SHOTCRETE CONSUMPTION WHEN WIRE MESH OR FIBRES



2A RECTANGULAR CROSS SECTION

2B CIRCULAR CROSS SECTION

FIG. 2 TYPICAL FIBRES USED IN SHOTCRETE WORK

7.5.5 *Polypropylene Fibres*

7.5.5.1 Polypropylene fibres may also be used to provide ductile behaviour to shotcrete. Certificate for declaration of performance of the fibre shall be obtained. *See* IS 16481.

7.5.5.2 Some of the important parameters of polypropylene fibres shall be:

a) Geometrical shape — Straight or deformed with plain or corrugated surface. Length of the fibres may be 40 mm to 60 mm. Equivalent diameter of the fibre shall be more than 0.30 mm.

b) Aspect ratio (length/equivalent diameter) — 60 to 75.

c) Ultimate tensile strength > 500 MPa.

7.5.6 Key to successful FRS construction is the use of a well trained and experienced shotcrete application crew. Pre-construction and post-construction testing of shotcrete shall be done for quality assurance as per IS 9012.

7.5.7 To increase the standup time, for a full front tunnel profile in poor rock quality condition (or squeezing rock conditions), spiling dowels are provided (*see* Fig. 3).

7.5.8 To stabilize the broken zone in squeezing ground conditions more than one layers of fibre reinforced shotcrete (FRS) is provided (*see* Fig. 4).

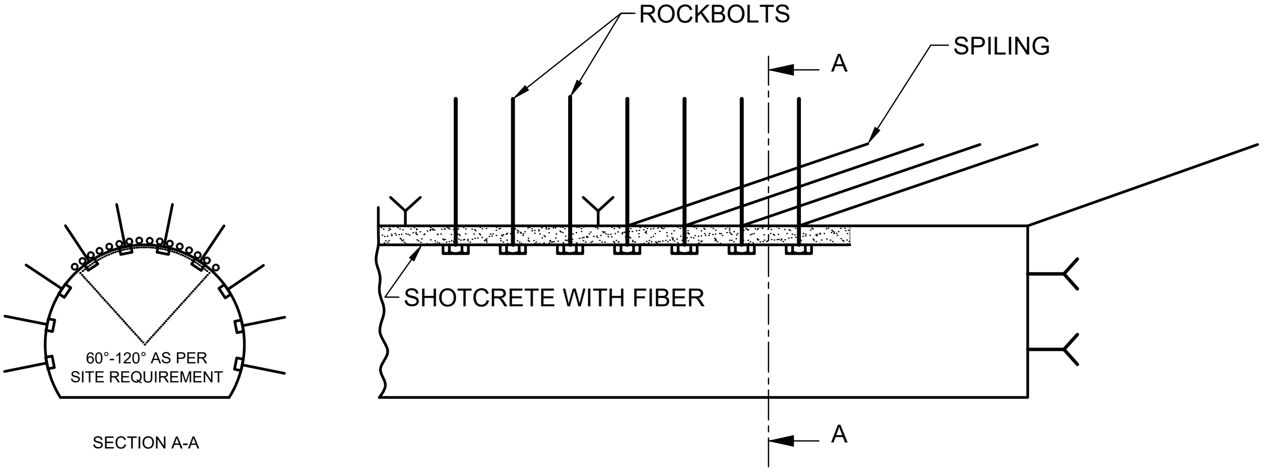


FIG. 3 ARRANGEMENT OF SPILING DOWEL WITH THE ADVANCEMENT OF TUNNEL FACE

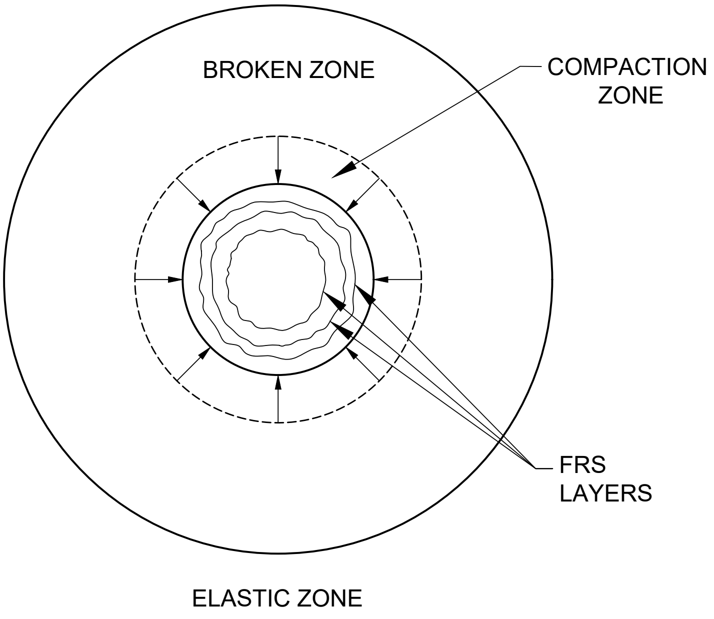


FIG. 4 STABILIZATION OF BROKEN ZONE IN SQUEEZING GROUND CONDITION

7.6 Capacity of Fibre Reinforced Shotcrete

7.6.1 It is assumed that the fibre reinforced shotcrete is intimately in contact with the rock mass and having the tendency to fail by shearing.

7.6.2 Capacity of fibre reinforced shotcrete is given by:

$$P_{\mathrm{fsc}} = \frac{2q_{\mathrm{fsc}}t_{\mathrm{fsc}}}{BF_{\mathrm{fsc}}} \qquad \ldots (2)$$

where

q_{fsc} = shear strength of fibre reinforced shotcrete (550 t/m^2),

t_{fsc} = thickness of fibre reinforced shotcrete (in m),

B = size of opening (in m),

BF_{fsc} = distance between vertical planes of maximum shear stress in FRS (in m),

F_{fsc} = 0.6 ± 0.05, and

P_{fsc} = support capacity of fibre reinforced shotcrete lining (in t/m^2).

7.6.3 The thickness of fibre reinforced shotcrete lining may be estimated by substituting ultimate support pressure (P_{roof}) in equation (2) in place of P_{fsc}. Additional layers of shotcrete should be sprayed to arrest tunnel closure if needed.

7.6.4 Proper equipment should be used to avoid bunching of steel fibres and to ensure homogeneous mixing of fibres in the shotcrete.

7.7 Drainage System in Road/Rail Approach Tunnels within Water-charged Rock Mass

7.7.1 Strips of about 500 mm width along the side walls and roof should not be shotcreted to allow free seepage of ground water otherwise shotcrete is likely to crack due to building up of seepage pressure behind shotcrete in heavily water-charged formations.

7.7.2 Drainage holes should be provided for proper drainage.

7.7.3 The catch drain should have adequate capacity to carry seepage water.

7.7.4 Water expandable friction bolts should be used to provide support to the rock mass as it is impracticable to put grouted bolts in heavy seepage conditions. However, suitably coated water expandable friction bolts may be used for enhanced durability.

8 TREATMENT OF SHEAR ZONE

8.1 The mean (*Q*) value may be determined, taking into consideration the breadth of weak/shear zone. The following equation (3) may be employed in calculating the weighted mean *Q* value from the *Q* values for shear zone and surrounding rock mass (*see* Fig. 5).

$$\log Q_{\mathrm{m}} = \frac{b \log Q_{\mathrm{wz}} + \log Q_{\mathrm{sr}}}{b+1} \qquad \ldots (3)$$

where

Q_{m} = mean value of rock mass quality *Q* for finding the support pressure [*see* IS 13365 (Part 2)],

Q_{wz} = *Q* value of the weak zone/shear zone,

Q_{sr} = *Q* value of the surrounding rock, and

b = thickness of the weak zone, in metre.

Weighted mean value of joint roughness number, j_{rm} may be obtained after replacing log *Q* by appropriate value of joint roughness number in equation (3). Similarly, weighted mean of joint alteration number, j_{am} may be calculated.

The strike direction θ and thickness of weak zone, *b* in relation to the tunnel axis is important for the stability of the tunnel and, therefore the corrected value b/(sin θ) has been suggested for the value of *b* in equation (3).

8.2 Special bolting system is required for supporting the weak shear zone. Figure 5 shows a typical treatment of a thin shear zone which is thicker than 500 mm.

8.3 First the gouge is cleaned out to the desired extent. Rock bolts are then installed across the shear zone and connected with chain wire-mesh. Finally, this 'dental' excavation is back-filled with shotcrete or gunite or fibre reinforced shotcrete. In wide shear zone (> 1 m), reinforcement has to be placed before shotcreting so that the reinforced shotcrete lining can withstand the heavy support pressure.

9 GUNITING

It is recommended that guniting with cement and sand mix of 1 : 3 to 1 : 4 may be used as a temporary support with or without wire mesh to prevent deterioration of rock surface. When used in combination with rock bolts and chain link fabric, it forms a permanent support.

10 ROCK BOLTS

10.1 General

Rock bolts can be broadly classified into two categories, active and passive. Active rockbolts are normally

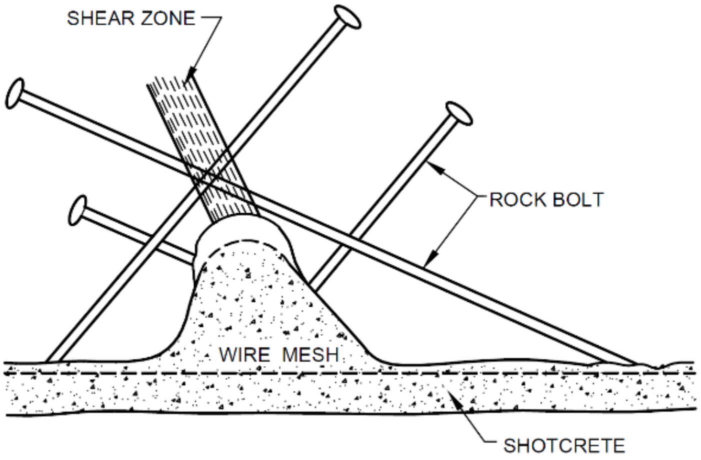


Fig. 5 Typical Treatment of a Narrow Shear Zone

end anchored into borehole by mechanical or other means and tensioned to provide active confinement to surrounding rock mass which acts as the reinforced rock arch. Passive rockbolts generally includes dowels/anchor bars or other arrangements which are not tensioned after installation and supports the loosened rock mass externally.

10.2 Types of Active Rock Bolts

10.2.1 *End Anchored*

10.2.1.1 *Wedge and slot bolt (mechanically anchored)*

These consist of mild-steel rod, threaded at one end, the other end being split into two halves for about 125 mm length. A wedge made from 20 mm square steel and about 150 mm long shall be inserted into the slot and then the bolt with wedge driven with a hammer into the hole which will force the split end to expand and grip the rock inside the hole forming the anchorage. Thereafter, a 10 mm thick plate of size 200 × 200 mm shall be placed, over which a tapered washer is placed and the nut tightened (*see* Fig. 6). The efficiency of the splitting of the bolt by the wedge depends on the strata at the end of the hole being strong enough to prevent penetration by the wedge end and on the accuracy of the hole drilled for the bolt. The diameter of such bolt may be 25 mm or 30 mm. Wedge and slot bolts are not effective in soft rocks.

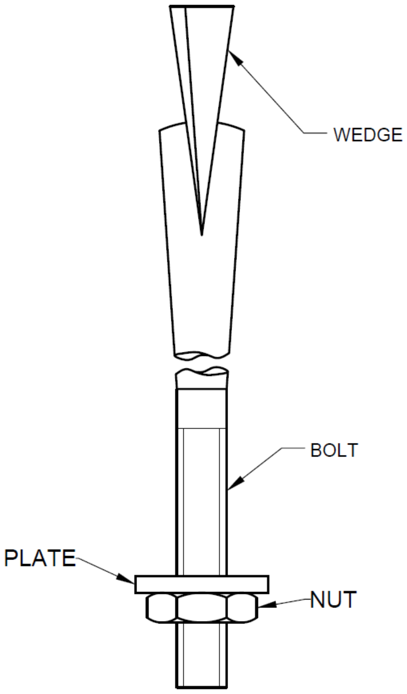


FIG. 6 WEDGE AND SLOT BOLT

10.2.1.2 *Wedge and sleeve bolt (mechanically anchored)*

This consists of a 20 mm diameter rod, one end of which is cold-rolled threaded portion while other end is shaped to form a solid wedge forged integrally with the bolt and over this wedge a loose split sleeve of 33 mm external diameter is fitted (*see* Fig. 7). The anchorage is provided in this case by placing the bolt in the hole and pulling it downwards while holding the sleeve by a thrust tube. Split by the wedge head of the bolt, the sleeve expands until it grips the sides of the tube. Special hydraulic equipment is needed to pull the bolts.

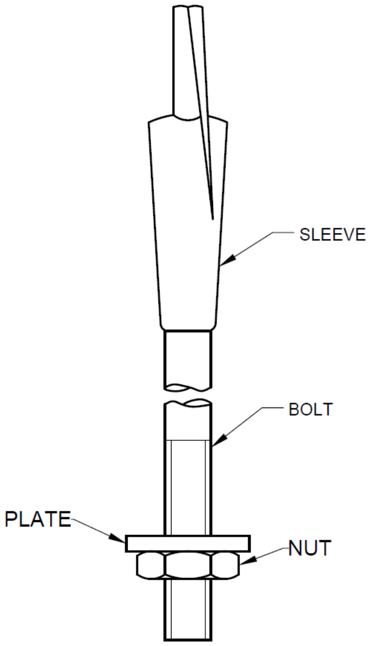


FIG. 7 WEDGE AND SLEEVE BOLT

10.2.1.3 *End grouted rebar bolt*

These consist of reinforcement bars encapsulated only in a limited length at the far end inside the borehole with resin grouts (*see* Fig. 8). These bolts can be pretensioned immediately after installation.

10.3 Types of Passive Rock Bolts

10.3.1 *Grouted*

10.3.1.1 *Fully grouted bolt (anchor or dowel bar)*

These consist of reinforcement bars and encapsulated in a borehole along its entire length with either cement or resin grout (*see* Fig. 9). They interact with the rock mass through mechanical interlock of the bolt surface with the grout agent. Usually cement grouted bolts are not pre-tensioned, due to risk of debonding between bolt and grout. Resin grouted bolt when pre-tensioned immediately after their installation act as active rockbolt. Typical yield load of these bolts can be up to 300 kN and depends upon diameter of the bolt.

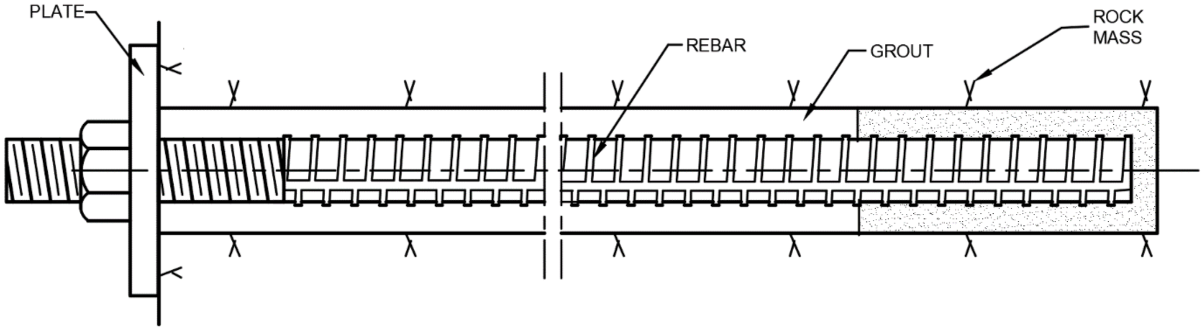


Fig. 8 End Grouted Rebar Bolt

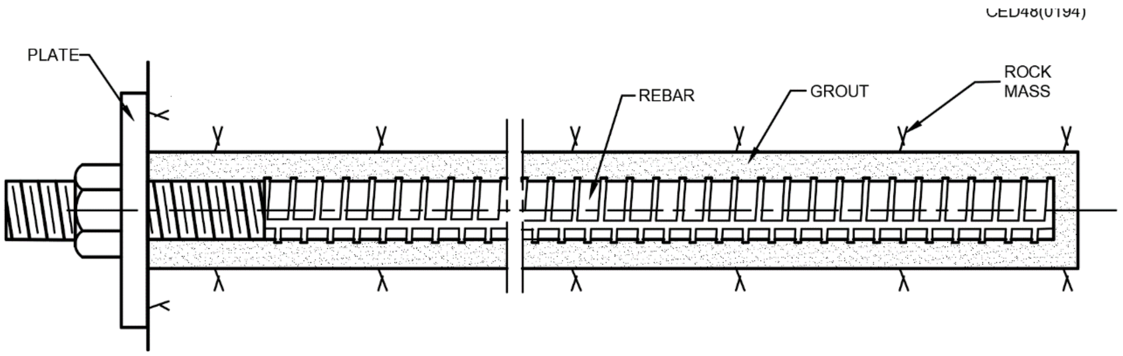


Fig. 9 Fully Grouted Rebar Bolt

10.3.1.2 *Self drilling bolt*

These bolts are used in conditions where borehole may collapse partially or completely before inserting the bolt or grout into the hole. It comprises a hollow anchor rod having coarse pitch threads on cylindrical surface of bolt and with a drill bit attached to its end (*see* Fig. 10). The bolt is drilled through a drilling machine and bit is sacrificed inside the hole. The cementitious grout is pumped into the borehole through hole in the anchor rod. It is important that anchor rod is sufficiently strong to cater to torsional and impact forces during installation. The bolt length can be extended by connecting coupling between two rods. The bolt is available in typical configurations of 25 mm outer diameter with yield load of 150 kN to 76 mm outer diameter having a yield load of up to 1 200 kN.

10.3.3 *Cable Bolts*

Cable bolts are used when reinforcement is needed to large depth, typically more than 10 m. These bolts are made of either single or multiple cable strands, and usually encapsulated with cementitious grout in boreholes. Each strand may consist of multiple wires (*see* Fig. 11). The advantage of cable bolts is the long reinforcement lengths and high load bearing capacity in the range of more than 1 000 kN. Deformation capacity of these bolts can be enhanced by debonding the bolt from grout by wrapping a plastic sleeve around the cable in the middle segment or upper segment near surface.

10.3.4 *Perfo Bolts*

This method of bolting consists of inserting into a bore hole a perforated cylindrical metal tube which is previously filled with cement mortar and then pushing a plain or ribbed bolt. This forces part of the mortar to ooze out through the perforations in the tube and come into intimate contact with the sides of the bore hole thus cementing the bolt, the tube and the rock into one homogeneous whole (*see* Fig. 12). The relation between the diameter of the bore hole and the diameter of perforated tube and bolts is given in Table 5.

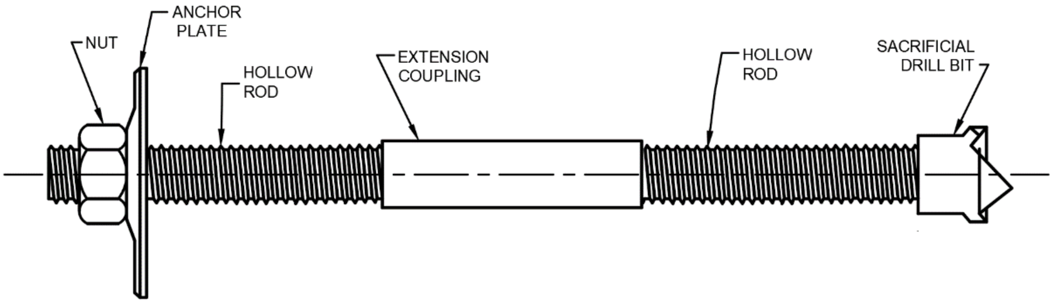


FIG. 10 SELF DRILLING ROCK BOLT

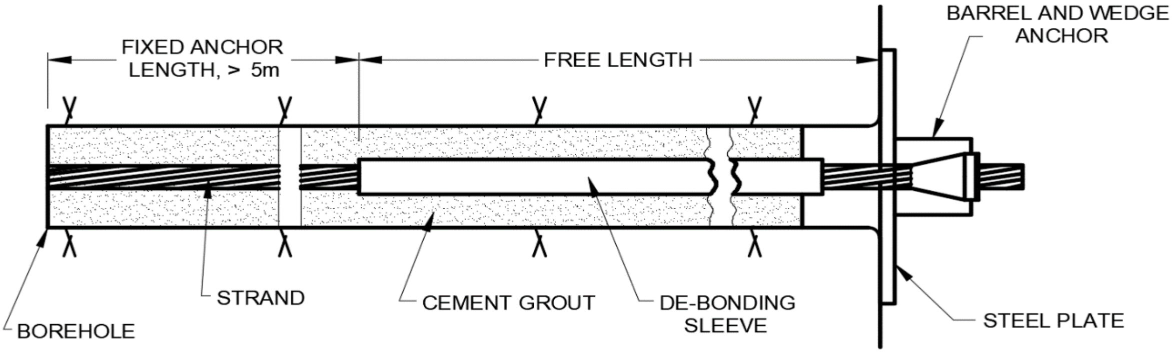


FIG. 11 CABLE BOLTS

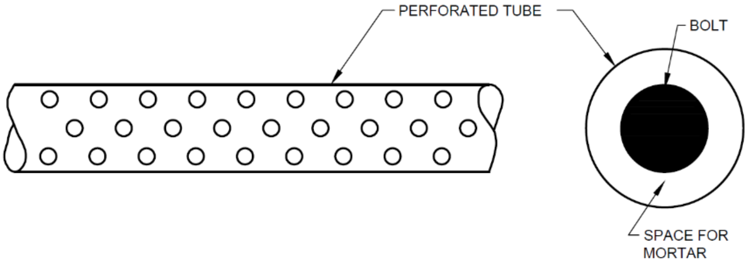


FIG. 12 PERFO BOLT

Table 5 Diameter of Perforated Tube and Bolts

(*Clause* 10.3.4)

Sl No.	Diameter of Bore Hole (mm)	Diameter of Perforated Tube (mm)	Diameter of Bolts (mm)
(1)	(2)	(3)	(4)
i)	40	36	30
ii)	38	31	25
iii)	31	27	18

NOTES

1 The bolts and anchors should be checked for their straightness within ± 1 mm.

2 Pull-out tests should be done on 5 percent of bolts and anchors to check their capacity (P_{bolt}).

10.3.5 *Water Expandable Friction Bolts*

These rock bolts are made of welded steel tube folded into omega shape in the cross-section (*see* Fig. 13). The bolt is placed inside a borehole and water is injected with high pressure of more than 25 N/mm^2 into folded tube through a small hole in the bushing at the bolt head. The tube expands and presses onto borehole wall resulting in a contact stress and mechanical interlock at bolt-rock interface. Oversize borehole diameter results in no contact and zero frictional strength. Typical breaking load of such bolts can be up to 240 kN. Advantages of these bolts are less installation time and immediate full load bearing capacity over entire installed bore length.

10.4 Design

10.4.1 Immediately after a tunnel has been advanced by a length '*t*' (*see* Fig. 14), the rock in this section expands and settles slightly developing a double arch effect. In the longitudinal direction of the tunnel, the arch rests on the still untouched rock at the front and on the already supported portion at the back (dashed lines in Fig. 14). The second arch effect, perpendicular to the axis of the tunnel is given by the form of the roof, which usually is an arch in tunnels. The period to which this combined arch will stand without support depends on the geological conditions, the length *t* and the radius of the tunnel roof. But in most cases, even in badly disintegrated rock, it will be possible to maintain this natural arch for some time, at least a couple of hours. If the natural arch is not supported immediately after mucking, it will continue to sink down slowly until it disintegrates.

10.4.2 The portion that is liable to fall is generally a parabolic arch in shape having a depth t/2 (*see* Fig. 14) though the loosening process will never go as deep as this if the movement is stopped by quick support. It is recommended that the bolts should not be made shorter than '*t*' that is twice the depth of presumed maximum loosening. The natural surrounding rock of the cavity is in this way transformed into a protective arch, the thickness of which is given by the length of the bolts '*l*' which should be bigger than '*t*', also should be between B/4 to B/3, as the arch also should have a certain relation to the width of tunnel. 'B' is the width of the tunnel.

10.4.3 The rock requires a prestress by bolting and the bolts should follow the static principles of prestressing in reinforced concrete as much as possible. As it is not possible to place bolts in the way of stress bars at the lower side of a beam, they should at least be given an oblique position in order to take the place of bent-up bars and stirrups (*see* Fig. 15).

10.4.4 With an arch instead of a beam, the shear forces will be greatly reduced by the vault effect but even in arch shaped roofs, shear forces may be caused by joint systems, especially by system of parallel layers like sedimentary formations, schist, etc. Hence the bolts should not only be made to exert a strong prestress to the rock but also should be set in a direction which suits best to the static demands of the geological conditions (*see* Fig. 15).

10.4.5 Just as a static member of prestressed concrete has to be prestressed before receiving the load, the rock also shall be prestressed by bolting before the load develops. This means that the space *t* (*see* Fig. 14) shall be bolted immediately after blasting while the next round is being drilled. The spacing between bolts/anchors should be less than half the length of bolts/anchors.

10.4.6 The pre-tension of ungrouted bolts is lost after blasting, so rock bolts shall be pretensioned again.

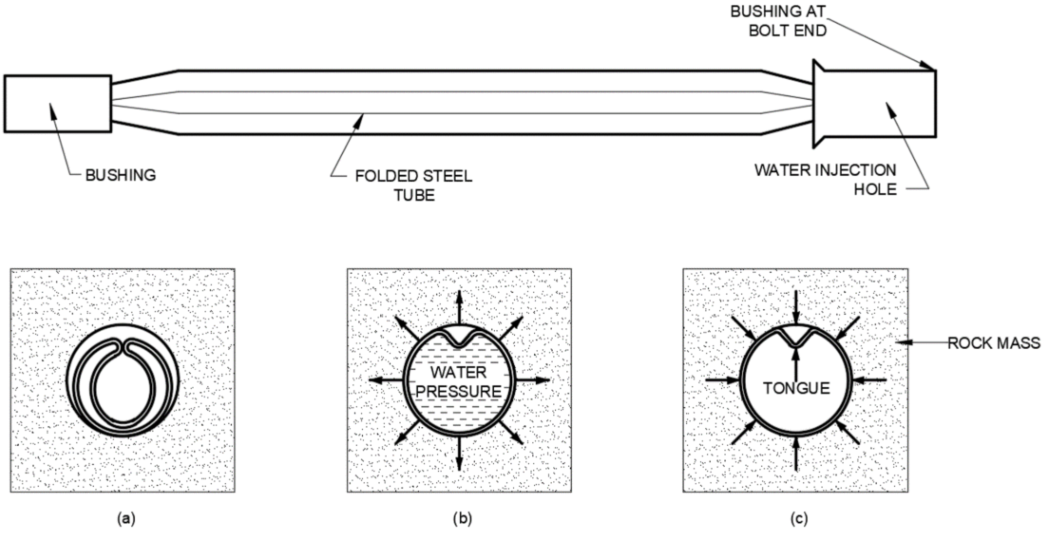


Fig. 13 Water Expandable Friction Bolt

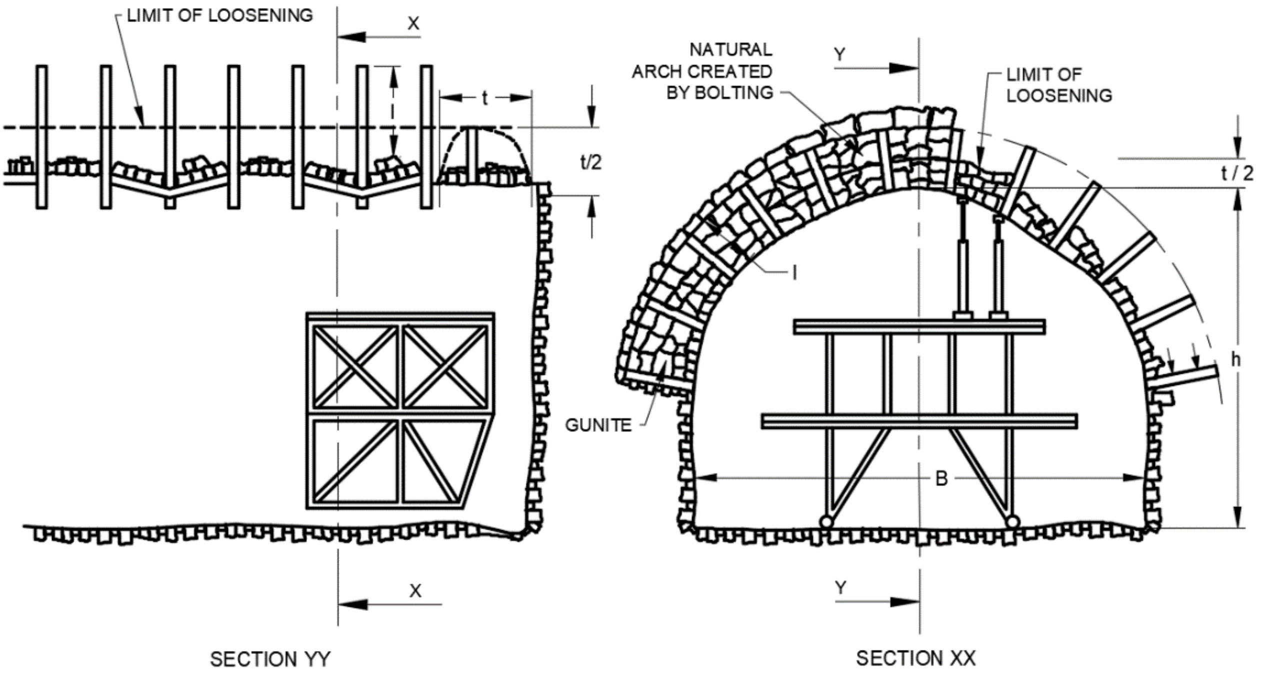
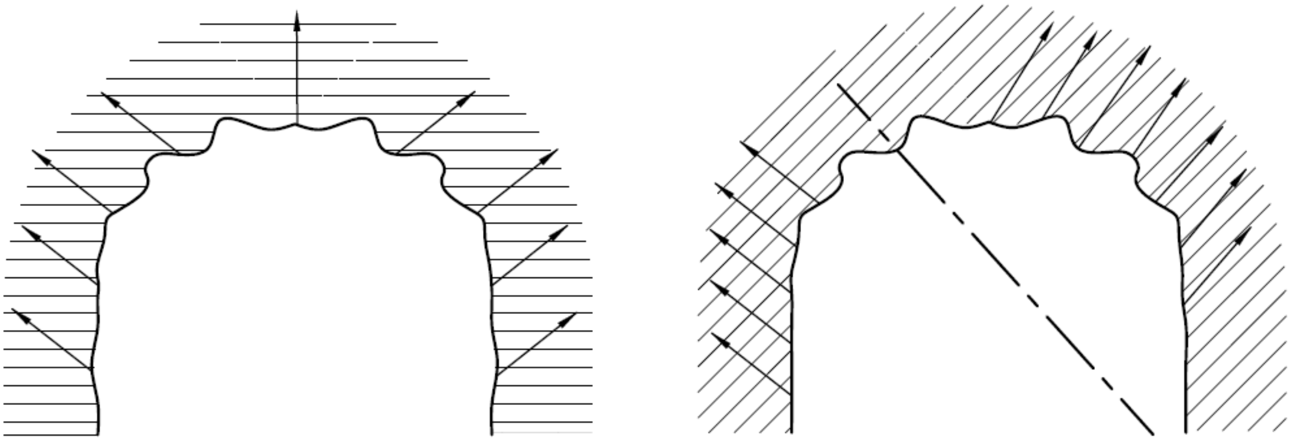
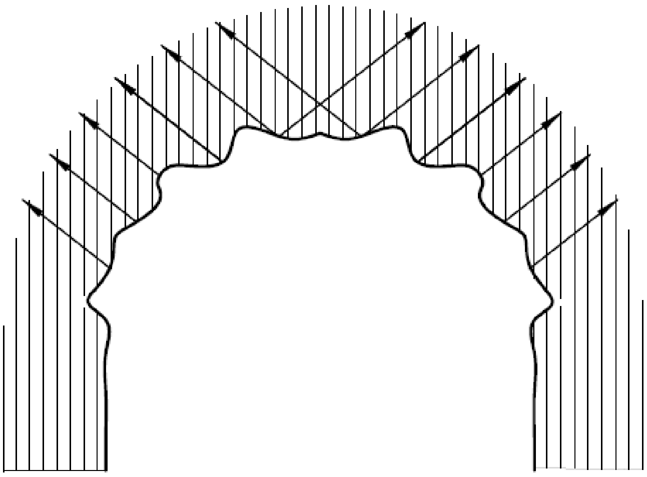


Fig. 14 Diagrammatic Section Demonstrating Of Roof Bolting



15A Horizontal Joint System

15B Inclined Joint System



15C Vertical Joint System

Fig. 15 Roof Bolting in Strata Running at Various Angle of Dip

10.5 Capacity of Rock Bolts/Anchors

10.5.1 The capacity of reinforced rock arch (P_{rrac}) is given by:

$$P_{\mathrm{rrac}} = \frac{2q_{\mathrm{crm}}\, l'\sin\theta}{B\, F_{\mathrm{s}}} \qquad \ldots (4)$$

where

$$\sin\theta = 1.3\, B^{-0.16} \leq 1;$$

The arch subtends an angle of 2θ at its centre.

q_{crm} = minimum uniaxial compressive strength of reinforced rock mass (joint will be critically oriented somewhere along the arch);

$$= \left[\frac{P_{\mathrm{bolt}}}{S^2_{\mathrm{bolt}}} - u\right]\left[\frac{1+\sin\phi_j}{1-\sin\phi_j}\right] \geq 0 \qquad \ldots (5)$$

$\tan \text{Ø}_{\mathrm{j}}$ = $J_{\mathrm{r}}/J_{\mathrm{a}}$ … (6)

P_{bolt} = tension in bolt or anchor capacity (in *t*);

S_{bolt} = centre-to-centre spacing of bolts/anchors (in m);

l' = effective thickness of reinforced arch (in m);

= $l_{bolt} - FAL/2 - S_{bolt}/2 + S_{rock}$ (in case of no shotcrete); … (7)

= $l_{bolt} - FAL/2 - S_{bolt}/4 + S_{rock}$ (in case of mesh reinforced shotcrete) …(8)

= $l_{bolt} - FAL/2$ (in both cases minimum value of l' to be checked) ...(9)

FAL = fixed anchor length (in m) of anchors to develop pull out capacity P_{bolt};

= 100 × diameter of anchor bars;

= $< l$ m in case of mechanically anchored;

S_{rock} = average spacing of fractures in rock (in m);

l_{bolt} = length of bolt or anchor (in m);

B = size of opening (in m);

F_s = mobilization factor of bolt/anchor;

F_s = $3.25\,P_{roof}^{0.1}$ (for mechanically anchored and pre-tensioned bolts) … (10)

F_s = $9.5\,P_{roof}^{-0.35}$ (for full column grouted anchors) … (11)

P_{roof} = ultimate support pressure in roof of tunnel (in t/m^2);

J_r = joint roughness number; and

J_a = joint alteration number.

The spacing and length of bolts should be so chosen that the estimated capacity of rock bolts/anchors, P_{bolt} is equal to ultimate support pressure, P_{roof}.

10.5.2 Full-column grouted bolts are more efficient in poor rock conditions according to equation 11 than ungrouted pre-tensioned bolts. For permanent supports, all bolts should be grouted.

10.6 Guidelines for Usage of Various Rock Bolts

Each type of Rockbolt has various merits and demerits. To tackle challenging ground conditions, most suitable type of Rockbolt should be used. Table 6 gives merits and demerits on usage various types of rockbolts.

11 LATTICE GIRDERS AND REINFORCED RIBS OF SHOTCRETE

11.1 Lattice girders (LG) and reinforced ribs of shotcrete (RRS) allow fast installation and facilitate shotcrete penetration through them. These supports minimize creation of projection shadow/void area in shotcrete lining, providing a composite structure acting as a reinforced lining. These supports can be fabricated before actual usage to different size and shape. Both LG and RRS also help to control excavated tunnel profile by permitting spiles/forepoles through them. Spacing of LG and RRS can be adjusted to meet required support stiffness and load carrying capacity.

11.2 Lattice Girders (LG)

11.2.1 Lattice girders (LG) consist of 3 or 4 primary reinforcement bars, arranged in triangular or rectangular manner, connected with stiffening elements which becomes integral part of tunnel shotcrete lining. Capacity of LG can be designed by selecting different bar sizes (*see* Fig. 16).

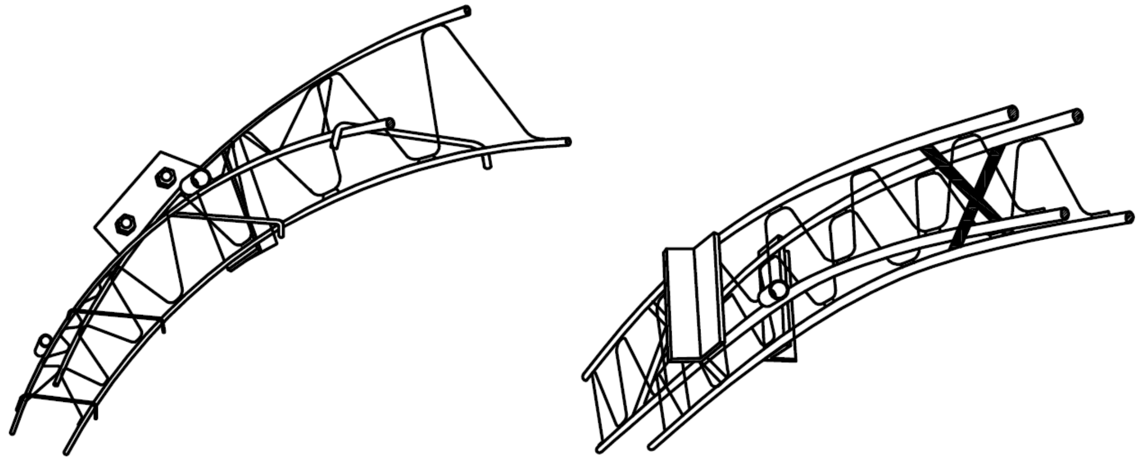


FIG. 16 TYPICAL ARRANGEMENT FOR THREE AND FOUR CHORD LATTICE GIRDERS

Table 6 Guidelines on Usage of Various Types of Rockbolts

(*Clause* 10.6)

Sl No. (1)	Type of Rock Bolt (2)	Merits (3)	Demerits (4)
i)	Mechanically anchored bolts	• Relatively inexpensive • Immediate support installation • Can be tensioned to provide confinement to rockmass	• Suitable for only moderately hard rock and low stress areas not prone to rock spalling • Low reliability • Regular monitoring and checking of tensioning • Prone to lose capacity with blast vibrations • Not recommended as permanent support unless coated suitably
ii)	Fully grouted rock bolt (resin)	• Preferred to use in upright position • Suits a variety of ground • Permanently effective • High shear capacity • Immediate activation • Can be used in holes with low ground water pressures	• Different resin manufacturer Installation guideline to be followed • Resin is expensive with limited shelf life • Stiff Rockbolts
iii)	Fully grouted rock bolt (cement)	• Suits a variety of ground • Permanently effective • High shear capacity • High load carrying capacity	• Not preferred for upright positions. • Time required for grout cure • Quality of mix affects capacity • Stiff Rockbolts • Can't be used in holes with ground water • Can't be used in ground with open fractures
iv)	Water expandable friction Bolts	• Immediate support installation • No grout chemical required • Suits a variety of ground	• Expensive • Corrosion prone in long run • Low shear strength • Requires separate water pump • Not recommended as permanent support unless coated suitably
v)	Self-drilling bolt	• Suitable for loose/fractured rockmass where hole wall is not stable • Grout pour through bolt hole ensuring complete length grouting	• Requires sacrificial bit suiting to hardness of rock • High steel grade to ensure higher torsional resistance of bolt for drilling without failure • Time required for grout cure
vi)	Cable bolts	• Very high load bearing capacity • Durable and permanent • Higher length installation > 10 m is easily possible	• Grout quality control is high • Can't be used in holes with ground water • Can't be used in ground with open fractures • Time required for grout cure before stressing

11.2.2 After scaling loose rock blocks and underbreaks, tunnel profile is shotcreted to attain profile of prefabricated LG. For thick shotcrete layers, a layer of wiremesh may be required to prevent loose falling of freshly applied shotcrete or for areas with low cohesion between rock and shotcrete. Afterwards, prefabricated LG is erected in place within permitted tolerances of line, level and spacing and covered with shotcrete up to half depth. Subsequently, spiles/forepoles are installed through empty area of LG, before embedding LG completely with shotcrete.

11.2.3 LG is not advisable for tunnel profile with large overbreaks, as they start carrying load only after bringing the tunnel profile to required geometry of prefabricated LG, which may take a lot of time to fill with grout/shotcrete.

11.2.4 In mild and moderate squeezing conditions, LG can be used effectively with lining stress controllers, to release tunnel pressure and reduce load on support.

11.3 Reinforced Ribs of Shotcrete (RRS)

11.3.1 RRS can be single layered, comprising 6 primary reinforcement bars of about 20 mm in diameter arranged in same plane at 100 mm to 125 mm spacing and tied to each other by a tie (usually cross reinforcement bars) at equal intervals (*see* Fig. 17); or double layered, comprising 2 additional reinforcement bars. These have a larger capacity to take load and moments. Due to its configuration, RRS is very flexible by itself and attains stiffness only after complete embedment with plain shotcrete. Moment capacity of RRS is usually analyzed as a reinforced shotcrete beam. Depending on length of reinforcement bars available at site and on the basis of excavated profile perimeter, RRS can be fabricated in two or up to three lengths, taking into account an overlap of approximately 1 m between any two lengths. Preferably the overlap in the reinforcement bars should be distributed so that it should not occur in the same cross section for all the bars. This is important for achieving good shotcreting results as many overlaps in the same cross section may cast a shadow behind the shotcrete. In order to increase capacity of the RRS, additional rock bolts can be installed on RRS near invert. For details of configuration and spacing of RRS for different rock quality, please see chart for support system [*see* IS 13365 (Part 2)].

11.3.2 After scaling loose rock blocks and underbreaks, thin layer of plain sealing shotcrete without fibres is applied to attain a smoothened rock surface. Thereafter, base layer of 100 mm to 150 mm is built up as per design on entire surface before installation of rock bolts at required spacing/pattern. Two additional rock bolts are installed on left and right tunnel wall near invert and at RRS chainage. Subsequently, reinforcement bars cage (up to 12 m in length) and comprising 6/8 rebars tied with tie-bar at 1 m interval, is erected and placed through protruding part of rock bolt and fixed using rock bolt face plate at each bolt location. Finally, entire RRS is embedded in shotcrete to provide atleast 50 mm cover to all reinforcement bars.

11.3.3 RRS provides efficient and cost-effective support mechanism through ground arch concept by acting through increased bearing surface at tunnel wall and acting together with rock bolts. RRS is beneficial for tunnel profile with large overbreaks, as the reinforcement bars cage of RRS can be easily bent inside the tunnel to match irregular tunnel profile, not requiring filling of gaps between wall and RRS unlike LG.

11.3.4 Using spiles/forepoles is comparatively difficult when the profile is irregular, however, overall time required to install RRS is much less and remaining profile can be filled up later. RRS can be erected behind the forepoles where it is possible to forepole in a rock quality that is better than the part of the weak zone around the tunnel periphery which RRS shall support. Usually, 32 mm diameter steel bolts with length of about 6-8 m are used as forepoles.

11.3.5 There is not enough experience in using RRS for cases with high overburden where moderate and heavy squeezing conditions are observed due to high stresses. For mild squeezing conditions, installation of RRS can be delayed based on observation.

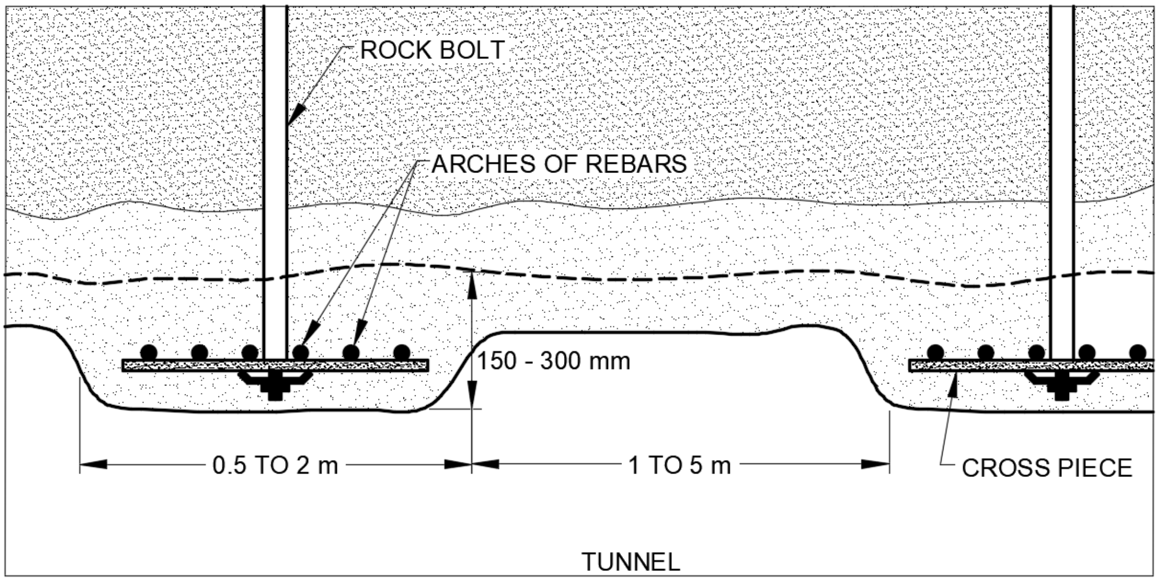


Fig. 17 Typical Arrangement for Reinforced Ribs of Shotcrete

12 STEEL RIBS

12.1 Rock tunnel support systems of steel may be generally classified into the following principal types:

a) Continuous ribs (*see* Fig. 18A),
b) Rib and post (*see* Fig. 18B),
c) Rib and wall plate (*see* Fig. 18C),
d) Rib, wall plate and post (*see* Fig. 18D),
e) Full circle rib (*see* Fig. 18E), and
f) Yielding arch steel rib with socketed joints.

Invert strut may be used in addition, with types (a) to (d) where mild side pressures are encountered (Fig. 18F) or squeezing ground is met.

12.2 Selection of Type of System

12.2.1 *General*

While choosing the type of support following factors shall be considered:

a) Method of attack;
b) Rock characteristics, its behaviour and development of rock load; and
c) Size and shape of the tunnel cross-section.

12.3 Selection of Supports with Reference to Surrounding Strata and Shape of Tunnel

12.3.1 *Continuous Ribs*

This type can be erected more rapidly than the other types and is generally recommended for use in rocks whose bridge action period is long enough to permit removal of gases and mucking. Invert strut may be used in addition where mild side pressures are encountered and squeezing ground is met.

12.3.2 *Rib and Post*

This type is generally recommended for use in tunnels whose roof joins the side walls at an angle instead of a smooth curve. It may also be used in large tunnels, such as double-track rail road or two-lane highway tunnels, to keep the size of the rib segments within handling and transporting limitations. Invert strut may be used in addition where mild side pressures are encountered and squeezing ground is met.

12.3.3 *Rib and Wall Plate*

This type is generally recommended for use in tunnels with a large cross-section with high straight sides through good rock or in large circular tunnels, where it is possible to support the wall plate by pins and where the strata below the wall plate does not require support. This type of support may also be used for tunneling through spalling rock, provided spalling occurs only in the roof. However, in many cases it is extremely difficult to establish adequate support for the wall plate at any point above the floor-line due to irregularity of the overbreak.

12.3.4 *Rib, Wall Plate and Post*

This type of support permits post spacing to be different from the rib spacing and is generally recommended for use in tunnels with high vertical sides. Invert strut may be used in addition, where mild side-pressures are encountered and squeezing ground is met.

12.3.5 *Full Circle Rib*

This type is recommended for use in tunnels in squeezing, swelling and crushed, or any rock that imposes considerable side pressure.

12.4 Spacing of Ribs

Spacing of ribs is given by:

$$S_{\text{rib}} = \frac{P_{\text{rib}}}{BP_{\text{roof}}} \qquad \ldots (12)$$

where

P_{rib} = steel rib capacity (in t),
S_{rib} = spacing of rib (in m),
B = size of opening (in m), and
P_{roof} = ultimate support pressure in roof of tunnel (in t/m^2).

The support capacity of steel ribs (P_{rib}) should be so chosen that the minimum clear spacing between ribs in poor rock condition is 100 mm more.

NOTE — Joints have not been shown in Fig. 18(A) to 18(F) and maybe located depending upon the construction and fabrication convenience.

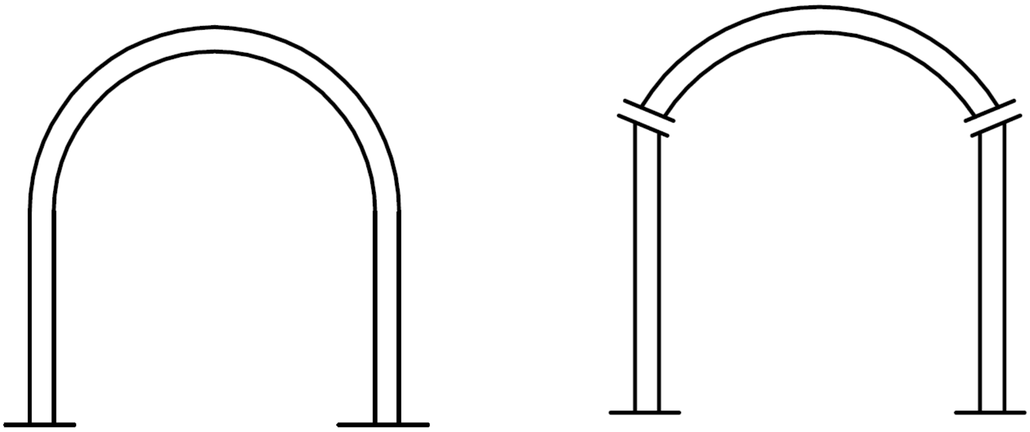
12.5 Selection of Steel Ribs Supports with Reference to Method of Attack for Tunnel

12.5.1 All the types of supports mentioned in **12.2** are suitable for the full-face method of attack for rock where required bridge action period for providing supports is available. Detailed guidelines are given in Table 2.

12.5.2 Rib and wall plate or rib, wall plate and post are suitable for heading and bench method. The rib, wall plate and post type may be supplemented by truss panels or crown bars, which are accessories developed to handle heavy loads that come quickly by supporting the intervening ribs while the bench is shot out.

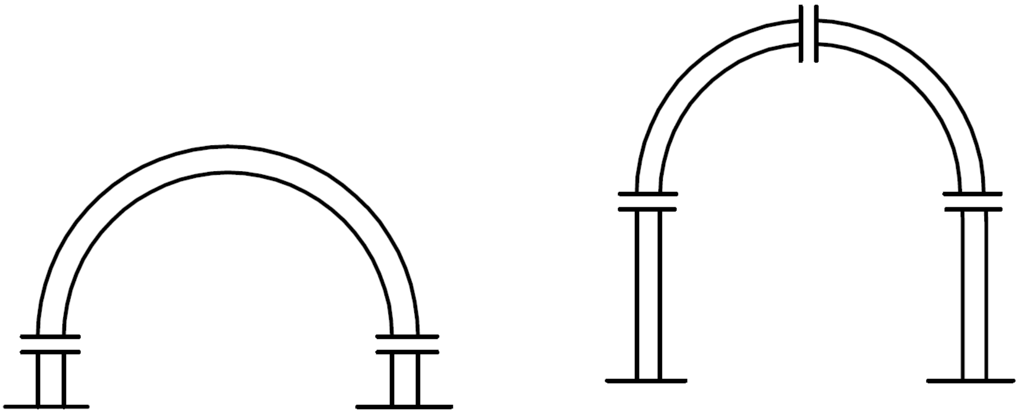
12.5.3 Where it becomes necessary to drive first the top heading only due to bad roof conditions, the rib and wall plate type of support is generally recommended for use in the heading, and post may or may not be used when the bench is taken out depending on rock conditions. If the post is used the excavated shape will be ‘D’ shaped while without the post the excavated shape will be a mushroom.

12.5.4 Where the side drift method is used for driving a large size tunnel in poor rock conditions, the rib, wall plate and post type of supports are recommended; the



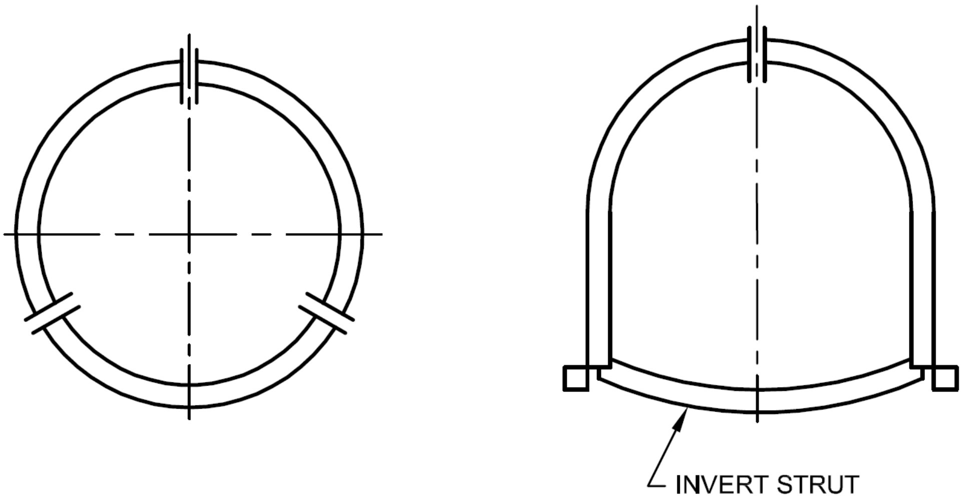
18 A CONTINOUS RIBS

18 B RIB AND POST



18 C RIB AND WALL PLATE

18 D RIB, WALL PLATE AND POST



18 E FULL CIRCLE RIB

18 F CONTINOUS RIB WITH INVERT STRUT

Fig. 18 Types of Steel Support Systems

wall plate, however, being flat. The posts and wall plates are erected in the drift which is driven ahead at each side at sub-grade (*see* Fig. 19).

12.5.5 Where extreme poor rock conditions are encountered, breakups to the crown may be made, leaving a central core. Temporary posts may be quickly placed between the core and the roof at dangerous spots, and crown bars may be slid forward to quickly catchup the roof. The roof ribs should then be placed on the wall plates and securely blocked to take the roof load, after which the temporary posts may be removed.

12.5.6 The side drifts themselves usually need support which should be removed just prior to shooting out the core of the main tunnel and reused ahead. The support system used for the drifts is hybrid. The outer side consists of the posts and wall plates which later becomes a part of the support for the main tunnel, whereas the inner side is a continuous rib (*see* Fig. 19).

12.6 Type of Support for Shafts

For shafts, usually the full circle rib or segmental ribs are recommended depending upon the slope and rock conditions. In vertical shafts, ribs may be hung from top by hanger rods and blocked and packed. The spacing of hanger rods may be worked out as in the case of tie rods keeping in view that they shall be strong enough to support the weight of ribs.

12.7 Components of Tunnel Supports

Design of various components of tunnel supports shall be done in accordance with IS 4880 (Part 6).

12.7.1 *Ribs*

Ribs may be made of structural beams. H-beams or wide flange beams should be preferred to I-beams, as the wider flanges provide more surface for blocking and lagging, and the section has greater resistance against twisting. Channel sections are not recommended as their asymmetrical section is prone to twisting, and their flanges are narrow. In small tunnels, however, channel bent about their minor axis may be used under ordinary loads. When choosing the steel section with different weights for arch posts, it is advisable to select beams of equal depth.

12.7.2 *Posts*

The spacing between the posts may be normally equal to that of the ribs. However, by inserting wall plate between the ribs and the posts the spacing of the posts can be made independent of ribs. The posts should be made of H-section. The depth of these should normally be the same as that of the ribs, though in many cases they may be of lighter sections as long as no side pressure is present.

12.7.3 *Invert Struts*

Where the side pressures are present and tunnel section has not been converted to a full circle, it is necessary to prevent the inward movement of the rib or posts feet and in such cases, invert struts should be provided at tunnel subgrade. They should be so attached to the vertical members that they receive the horizontal pressure. They may be curved to form an inverted arch where there is upthrust from the floor.

12.7.4 *Wall Plates*

12.7.4.1 The following types of wall plates are commonly used:

a) Double beam,
b) Single beam, and
c) Flat.

12.7.4.2 The double and single beam wall plates which are intended to resist bending in vertical planes are recommended for use to transmit the loads from the ribs on to block or posts with a spacing different from that of the ribs. Flat wall plates merely serve as an erection expedient and a convenient surface for horizontal blocking, their resistance to bending in vertical planes being very small. Whenever flat wall plates are used, a post shall be placed under each rib.

12.7.4.3 Double beam wall plates may be made of two I-beams placed side by side, webs vertical with about a 100 mm space between flanges to give access to the clamping bolt and admit concrete (*see* Fig. 20). The beams should be spaced by vertical diaphragms welded under each rib seat. Ribs and posts should be clamped by toggle plates and bolts, thus avoiding the time required for matching bolt holes. This method of attachment also permits variable spacing of either or both the ribs and the posts. This type of beam provides a broad surface of contact for blocking and to engage ribs and posts. Its box section makes it stable with respect to rolling and twisting.

12.7.4.4 Single beam wall plates may be H-beams, with web vertical. To enable them to transmit vertical loads from rib to post, they may be reinforced at each rib seat with vertical T-shaped plates, if necessary (*see* Fig. 21). Attachment of ribs and posts shall be made by bolting through the flanges.

12.7.4.5 Flat wall plates may be I-beams or wide flange beams used with their webs horizontal. They function merely as a cap for the posts and a sill for erecting roof ribs. The web shall be punched with vent holes which also helps to pass reinforcing rods if the concrete is reinforced.

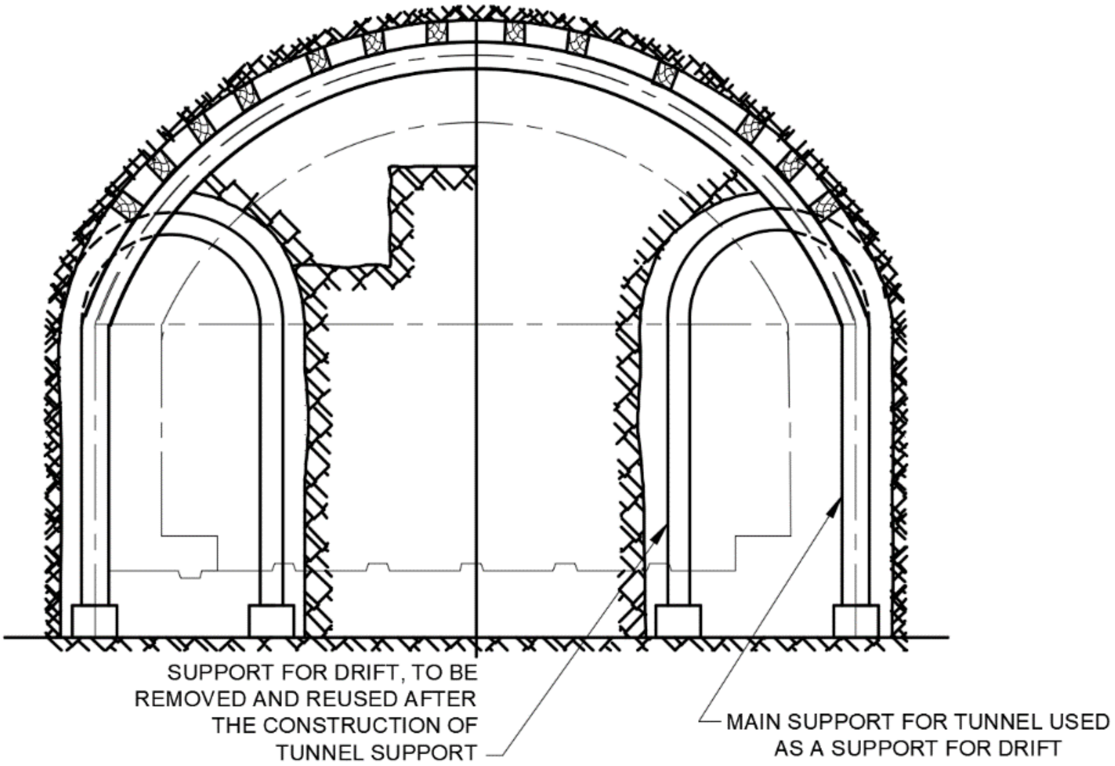


Fig. 19 Side Drift Method

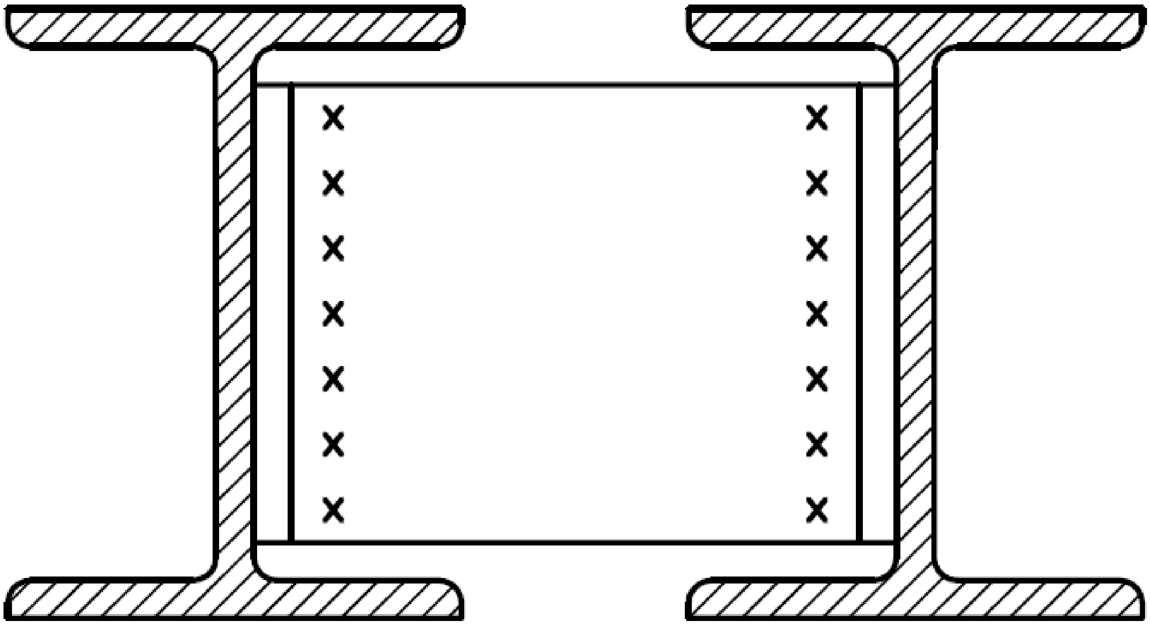


Fig. 20 Double Beam Wall Plate

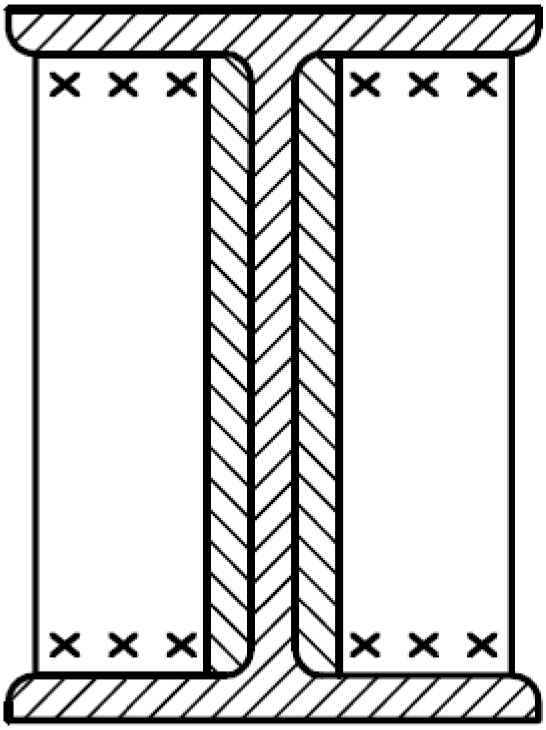


FIG. 21 SINGLE BEAM WALL PLATE

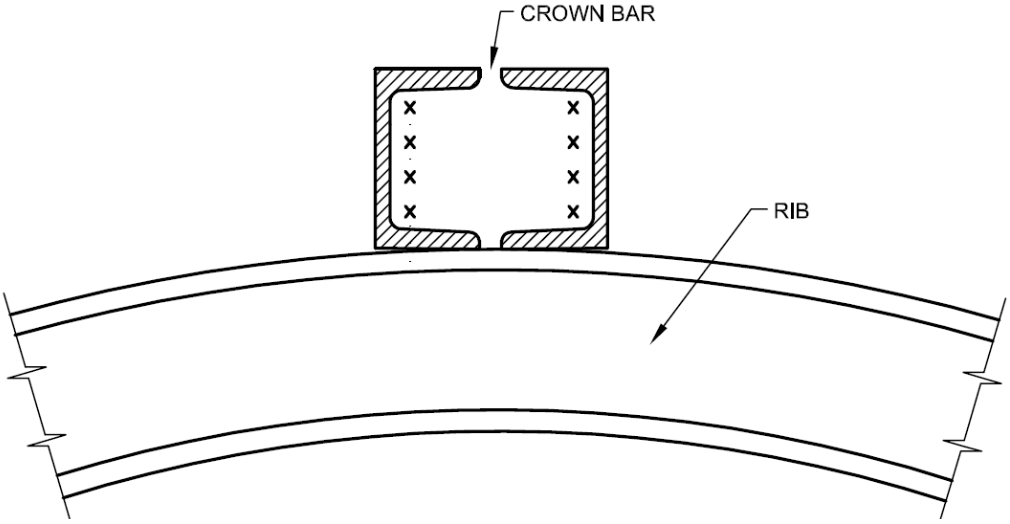
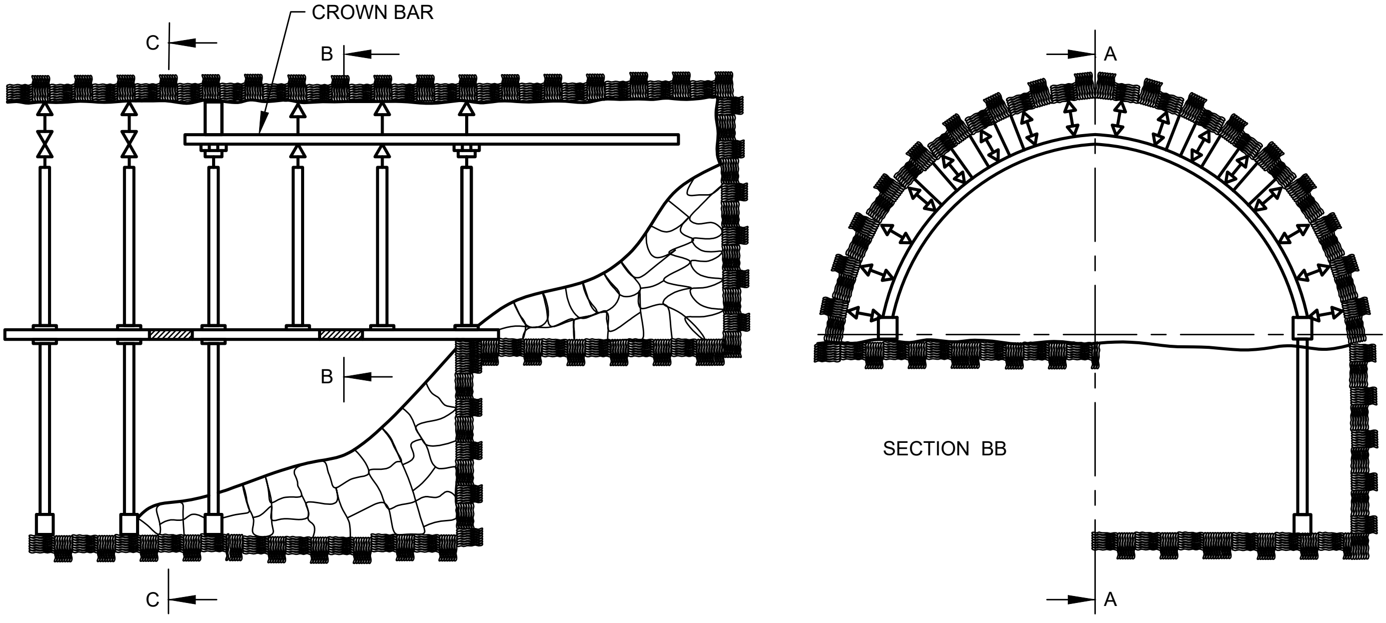


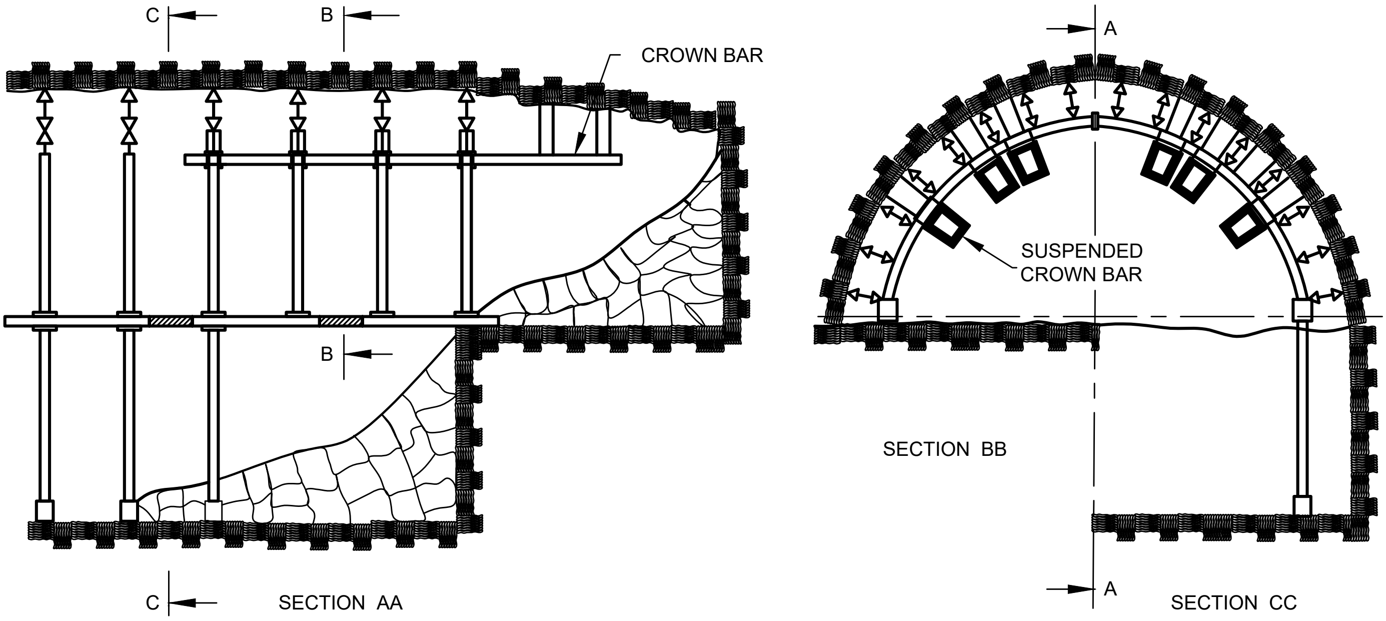
FIG. 22 CROWN BAR

12.7.5 *Crown Bars*

12.7.5.1 Crown bars may be built up of double channels (*see* Fig. 22) or may be H-beams or square timber beams. They are located parallel to the axis of the tunnel either resting on the outer flanges of the ribs already erected (*see* Fig. 23A) or attached to the ribs in hangers (*see* Fig. 23B). Crown bars are an accessory, a construction expedient intended to carry loads till the rib sets are erected and the loads permanently transferred to them. They have one of the two functions to perform: (a) to support the roof immediately after ventilation and thereby gain time for the installation of ribs, or (b) to support the roof or roof ribs over the bench shot thereby relieving or supplementing the wall plates.



23A Crown Bar Supported Over Ribs



23B Crown Bars in Hangers

⇕ BLOCKING BETWEEN ROCK AND RIB

▯ BLOCKING BETWEEN CROWN RIB AND ROCK RIB

Fig. 23 Method of Support for Crown Bars

12.7.6 *Truss Panels*

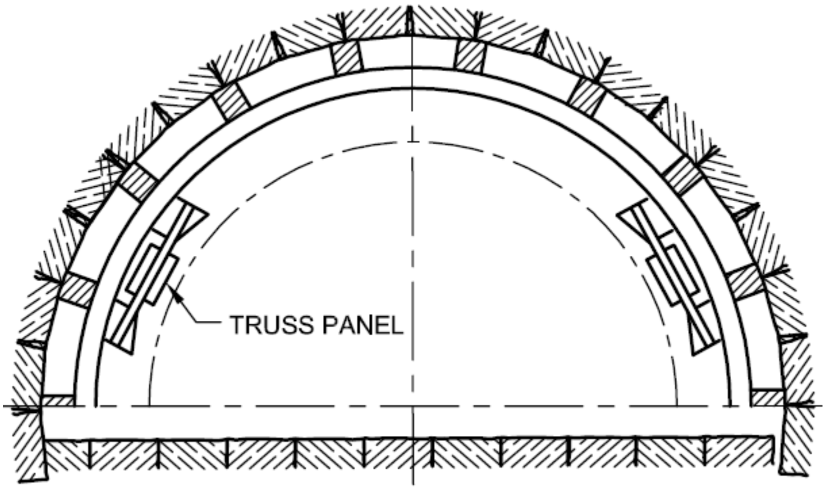
12.7.6.1 These are accessories for use with the combination of rib and post types of support, for the heading and bench or top-heading methods of attack and heavy roof loads. Their purpose is to form, in combination with the ribs, a truss to span the gap produced by the bench shot.

12.7.6.2 The truss panels should be attached to the inside face of the ribs for a distance of one or more ribs ahead of the bench shot (*see* Fig. 24) and should be left there until posts are installed, at that time they should be removed and sent up ahead. Attachment should be by means of only two bolts at each rib. The truss thus formed may even be designed to carry the roof over two bench shots making it more convenient to get in the post.

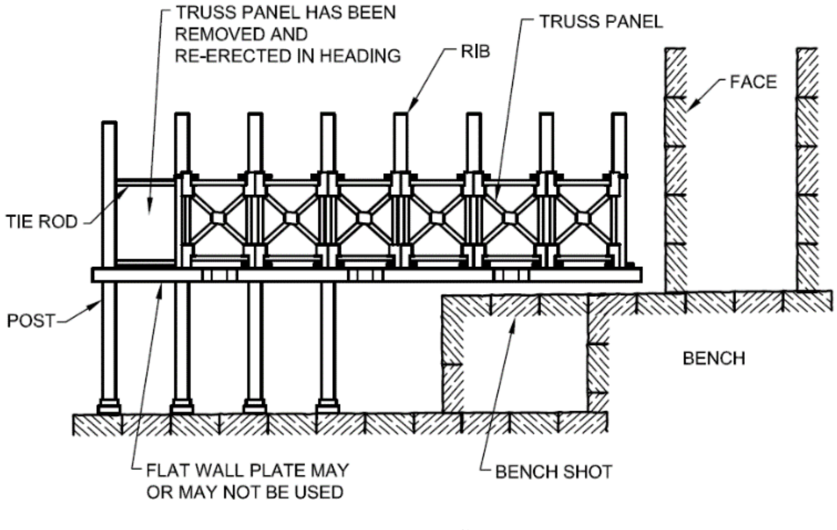
12.7.6.3 When truss panels are used, no wall plates are required although the flat wall plate may be used to keep the lower ends of the ribs lined up laterally if it is difficult to block the individual ribs against the rock. The truss panels eliminate the need for wall plate for drifts.

12.7.7 *Bracing*

12.7.7.1 Longitudinal bracing or struts increase the resistance of ribs and posts to buckling about their minor axis and prevent a displacement of these set members during blasting. If the space between the ribs or post is bridged by lagging which is firmly attached to the webs, no such bracing is required. The most common types of bracing are the rods and collar braces (*see* Fig. 25). The braces may, however, be placed as convenient between the rods.



CROSS – SECTION


TRUSS PANEL HAS BEEN REMOVED AND RE-ERECTED IN HEADING
TRUSS PANEL
RIB
FACE
TIE ROD
POST
BENCH
FLAT WALL PLATE MAY OR MAY NOT BE USED
BENCH SHOT

LONGITUDINAL SECTION

FIG. 24 TRUSS PANEL

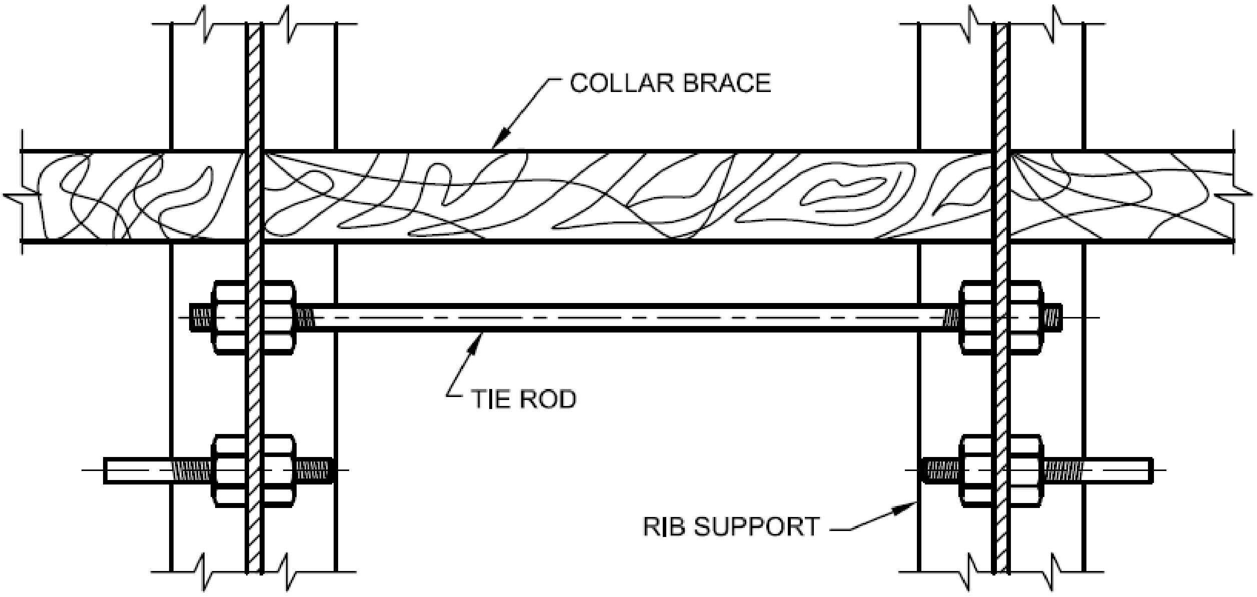


FIG. 25 COLLAR BRACE AND TIE RODS

12.7.7.2 Tie rods may be 15 mm to 20 mm rods, with thread and two nuts on each end. The length shall be at least 100 mm more than the spacing of the ribs. The spacing of the rods shall be kept such that slenderness ratio l/r for ribs is not greater than 60, where l is the spacing of the rods, and r is the least radius of gyration of the ribs. Collar braces may be usually pieces of timber, 75 mm × 100 mm, 100 mm × 150 mm, 150 mm × 150 mm or any conventional size. Holes in pairs shall be provided in the web of ribs and posts for the tie rods. Collar braces shall be set in the line between ribs, tie rods inserted and the nuts tightened. Wooden collar braces should be removed before placing final lining.

12.7.7.3 Spreaders (*see* Fig. 26) which are additional braces may be angles, channels, or I-beams with a clip angle or plate either bolted or welded on each end to the ribs. These are left in the concrete. In tunnels having steep slopes tie rods may be replaced by spreaders.

12.7.8 *Blocking*

It is generally done by using timber pieces tightly wedged between the rock and the rib. These timber pieces are to be removed prior to concreting.

12.7.9 *Lagging*

12.7.9.1 It performs one or more of the following functions:

a) To provide protection from falling rock or spalls;
b) To receive and transfer loads to the rib sets;
c) To provide a convenient surface against which to block in case it is not convenient to block directly against the rib, because of irregular overbreak;
d) To provide a surface against which to place back packing;
e) To serve as an outside form for concrete lining, if concrete is not to be poured against the rocks; and
f) To divert water, and to prevent leaching and honey-combing of concrete.

12.7.9.2 Lagging may be either of steel, precast concrete or timber. Steel laggings may be made out of channels, beams, beams and plates and liner plates (*see* Fig. 27 and 28). Liner plates, which are pressed steel panels may also be used with or without ribs depending upon the rock conditions. It is recommended that use of timber in underground work should be minimized as far as practicable, since timber once fixed can be rarely removed safely and is likely to deteriorate and prove to be a source of weakness.

12.7.9.3 The spacing of lags shall be closest at the crown, increasing down to spring line. On the side only an occasional lag should be used, if necessary. Close lagging should be employed where rock conditions make it necessary.

12.7.10 *Packing*

The function and type of packing depends on the rock condition. In dry tunnels through jointed rock, packing is only used to fill large cavities produced by excessive overbreak. In broken, crushed or decomposed rock it serves to transfer the rock load to the lags, thereby acting as a substitute for excessive blocking. In squeezing rock it provides continuous contact through the laggings with the rib sets. In jointed water-bearing rock it has primarily the function of a drain.

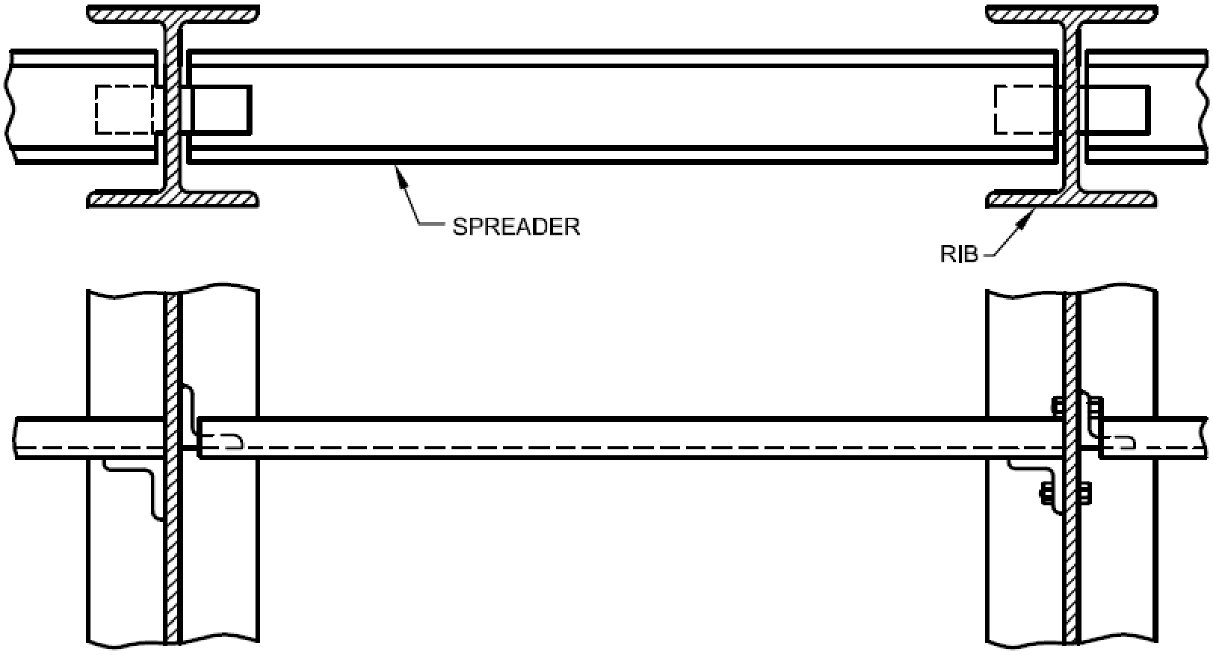


Fig. 26 Spreader Details

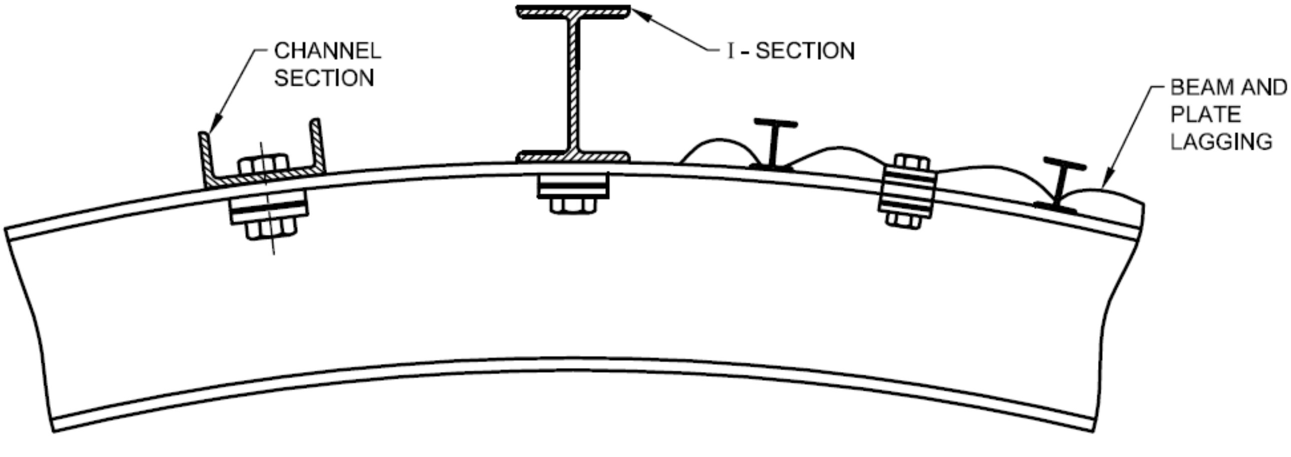


Fig. 27 Types of Lagging

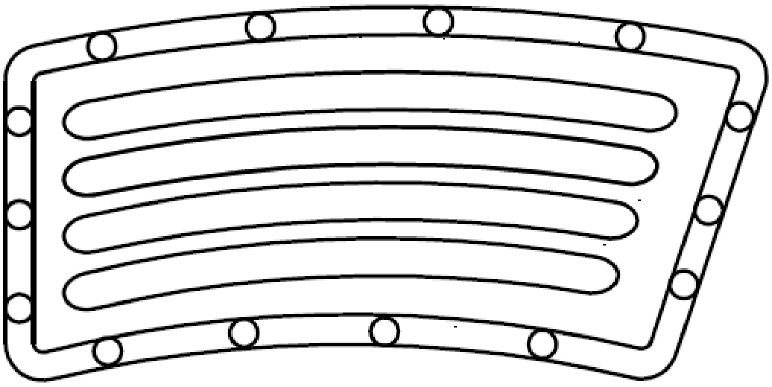


Fig. 28 Liner Plate

12.7.11 *Dry Packing*

Dry pack, which usually consists of tunnel spoil (hard) shovelled or hand packed into the space between the lagging and the rock, is recommended for use only where excessive rock loads are not likely to develop.

It should be placed simultaneously with the erection of the lagging. Starting at the lowest point, a few lags should be placed and tunnel spoil (hard) shovelled in behind. This procedure should be carried up to the crown at which point it is necessary to pack endwise.

12.7.12 *Concrete Packing*

It is recommended for use where considerable rock loads are anticipated. However, its use is not recommended in case the tunnel supports are designed as yielding supports. Concrete packing may be M 10 concrete conforming to IS 456. It may be placed by manual labour or by pneumatic placer to the extent possible. Where excessive loads are anticipated, concrete packing should start from the inner flanges of the steel support so as to embed the whole steel supports in concrete. In such cases it is recommended that precast concrete may be used as additional lagging between two adjacent ribs so as to serve the purpose of formwork.

12.8 Factors Determining Spacing and Layout of Supports

12.8.1 The strength and spacing of rib system shall be determined by rock load. For a given rock load and cross-section of tunnel, the spacing between the ribs shall be worked out. The spacing of the ribs should be so chosen that the sum of the cost of ribs and lagging is minimum. For preliminary designs in ordinary rocks the depth or rib section may be taken as 60 mm to 75 mm for every 3 m of bore diameter with ribs spaced at about 1.2 m for moderate loads, 0.6 m to 1.0 m for heavy loads and 1.6 m for very light loads. Whether the ribs shall be of two or more pieces will depend on the lengths of the structural steel members available.

12.8.2 For junctions, plugs and control chamber, etc, supports shall be designed to suit special features of the work and its construction procedures.

12.8.3 Concrete blocks of suitable size and thickness may be provided, if necessary, below the vertical lags to provide adequate bearing area to the rib.

12.8.4 In tunnels, where supports are not to be used as reinforcement, they may be installed plumb or perpendicular to the axis of the tunnel depending on tunnel slope and as found convenient. However, where supports are to be used as reinforcement in pressure tunnels, they may be installed at right angles to the tunnel axis, if practicable.

12.8.5 For speedy erection of supports, it is suggested to:

a) design support system with a minimum number of individual members, consistent with construction convenience;
b) design the joints with utmost simplicity and with minimum number of bolts; and
c) check clearances for bolt and wrench, to be used at site, during fabrication of the members so that loss of time during support erecting can be minimized.

12.9 Design

12.9.1 *General*

The design of steel components for tunnel supports shall generally conform to IS 800.

12.9.2 *Stresses*

Permissible stress in steel shall be in accordance with IS 800.

12.9.3 *Ribs*

Rock load may be assumed to be transmitted to the ribs at blocking points, each blocking point carrying the load of the mass of rock bounded by four planes, namely, the longitudinal planes passing through midpoints between the blocks and transverse planes passing through mid-points between the ribs to a height equal to the acting rock load. The blocking points may be assumed to be held in equilibrium by forces acting on it in the same manner as panel points in a truss. Values of thrust in the rib may be computed by drawing the force polygon. Ribs shall be designed for the thrust thus computed taking into account the eccentricity of this thrust with reference to the rise of the arc between the blocking points which will cause flexural stresses in addition to direct stresses.

12.9.4 *Tie Rods*

Tie rods are threaded 15 m to 20 m rods, which are used to hold and maintain spacing between adjacent steel ribs. [*see also* IS 5878 (Part 4)].

12.9.5 *Lagging*

Lagging between steel ribs for tunnel support may be designed for the load of rock mass as shown in Fig. 29 [*see also* IS 5878 (Part 4)].

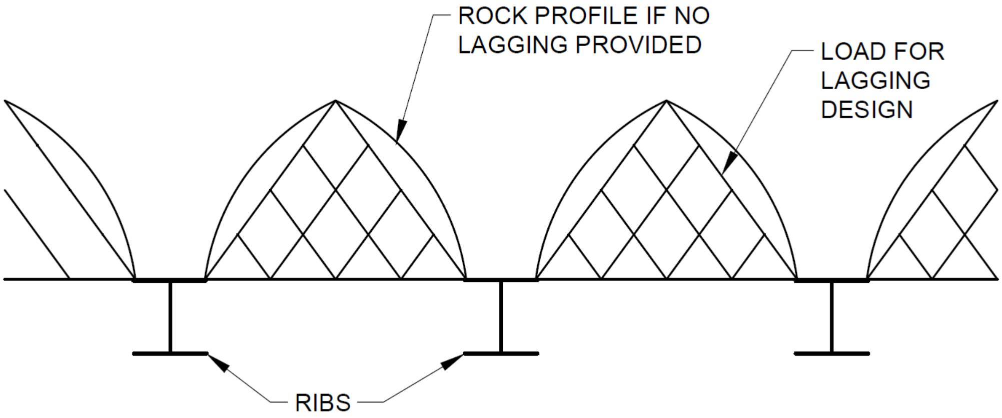


FIG. 29 LOADING DIAGRAM FOR LAGGING

12.9.6 *Liner Plates*

Where only liner plates are used for support their cross sectional area and their joints shall be designed to transmit the thrust (*see* **12.9.3**). It shall be ensured that liner plates are thoroughly in contact with the ribs so that passive resistance is developed and no bending moments are induced. For tunnels with more than 3 m diameter liner plates may be reinforced by I-beams. Where liner plates do not form a ring and are used in top half ribs, they shall be designed as lagging [*see also* IS 5878 (Part 4)]. The thickness of liner plates may vary from 5 mm to 10 mm depending upon the size of bore and loads encountered. The diameter of bolts may vary from 12 mm to 15 mm.

12.9.7 *Joints*

Butt joints should be preferred to spliced joints. In soft grounds and poor rock, welding of joints in the field should be avoided as far as possible.

13 GROUTING

13.1 General

Grouting is carried out to fill discontinuities in the rock by a suitable material so as to improve the stability of the tunnel roof or to reduce its permeability or to improve the properties of the rock. Grouting is also necessary to ensure proper contact of rock face of the roof with the lining. In such cases the grouting may be done directly between the two surfaces or the process of grouting may be used to fill the voids in the rubble packing where used. All the three types of grouting may not be required in all cases. The grouting procedures should aim at satisfying the design requirements economically and in conformity with the construction schedules. The basic design requirement generally involve the following:

a) Filling the voids, and cavities between the concrete lining and/or rock and/or between the concrete and steel liner;
b) Strengthening the rocks around the bore by filling up the joints in the rock system;
c) Strengthening the rock shattered around the bore;
d) Strengthening the rock prior to excavation by filling the joints with cementing material and thus improving its stability; and
e) Closing water bearing passages to prevent the flow of water into the tunnel and/or to concentrate the area of seepage into a channel from where it can be easily drained out.

13.2 Before drawing up the specifications for grouting, the design requirements shall be established. In general for all underground structures, grouting is a universal requirement for all concrete lined tunnels. Design requirements are only to establish the maximum allowable pressure at which this grouting is to be carried out and the zone in the cross-section and the spacing of grout holes, both in the direction of the tunnel. For consolidation grouting, the design requirement to be established is the thickness of the rock stratum around the bore that is to be strengthened and made impermeable, the pressure and the spacing pattern of holes. This will determine the depth to be grouted.

13.3 For tunnels, the commonly used procedures are to continue grouting to refusal at the design pressure in each hole or to interrupt the grouting if there is heavy intake with little or no pressure build up, indicating very open structures and escape of grout over a long distance.

13.3.1 *Pattern, Depth and Arrangement of Holes*

13.3.1.1 *Backfill or contact grouting*

13.3.1.1.1 The purpose of backfill grouting is to fill the space left unfilled with concrete between the concrete lining and the rock surface in the arch portion of any tunnel or cavity due to shrinkage of concrete.

13.3.1.1.2 Backfill grouting should be done after the concrete in lining has gained strength. The period of waiting may be from 21 days to 28 days. In case of precast lining segments this restriction of waiting will not apply and the grouting may be done immediately after the segments are erected.

13.3.1.1.3 Backfill grouting is limited to the arch portion of a tunnel or cavity and is not required in case shaft of the concrete is poured in a concrete ring.

13.3.1.1.4 The grout holes at the crown should be placed 5° to 10° from the crown, alternately to the left and right of the crown. In addition to the crown hole there shall be two more holes, one on either side of the crown. These holes will be 90° apart and will be located such that one of these two holes is at 22.5° from the crown alternately on the right and left of the crown. Such sections shall be, normally, 3 m apart. The exact location of the holes may be varied or additional holes provided depending upon the actual excavation profile at any section. The exact spacing of sections may also be varied on similar considerations. It should, however, be also adjusted to suit the length of the arch shutter used in such a way that there is no hole at the joint and the normal pattern of holes is more or less uniform in the shutter length.

13.3.1.1.5 In the case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 450 mm to 600 mm, above the junction of the invert and side walls or overt.

13.3.1.1.6 The mortar used for backfill grout shall normally consist of cement, sand and water mixed in the proportion of 1 : 1 : 1 by weight. It may, however, be suitably modified if conditions so warrant. The size and grading of sand should be determined for each job by actual experimentation as it would depend on the type of sand and equipment available.

13.3.1.1.7 Backfill grouting should normally be done at a pressure of 0.2 N/mm^2.

13.3.1.2 *Contact grouting*

13.3.1.2.1 The aim of contact grouting is to fully pack the space between the concrete lining and the rock surface or the space between the steel liner and concrete lining caused by shrinkage or left unfilled even after backfill grouting. This is required for fulfilling the design assumption of the rock/concrete taking part of the load along with the lining and to prevent local accumulation of water, if any, and building up local pressure.

13.3.1.2.2 Contact grouting should be done after the concrete lining has gained strength to withstand the pressure and shrinkage, if any has taken place. The usual minimum period of 25 days to 28 days of waiting should be allowed.

13.3.1.2.3 The contact grouting should be limited to only the top arch (90° on either side of the crown) of tunnels. In case of vertical shafts and steel liner, contact grouting should be done along the full periphery. In case of steel liners, the grouting should be done usually at specific points as recommended in IS 5878 (Part 6).

13.3.1.2.4 The holes at the crown shall be placed 5° to 10° from the crown, being alternately to the left and right of the crown. In addition to the crown hole, there shall be two more holes one on either side of the crown in each section. These holes will be 90° apart and will be located such that one of the two holes is at 22.5° from the crown, being alternately on the right and left of the crown. Such holes shall normally be 3 m apart.

13.3.1.2.5 In case of circular or horse-shoe tunnels, in addition to these holes, two holes (one on either side), located roughly at 45° on either side of the invert should be used. The location should be such that the holes are about 450 mm to 600 mm above the junction of the invert and arch.

13.3.1.2.6 The depth of holes for contact grouting shall be such that at each location, the holes extend 300 mm beyond the concrete lining into rock.

13.3.1.3 *Consolidation grouting*

13.3.1.3.1 The aim of consolidation grouting is to fill up the joints and discontinuities in the rock up to the desired depth.

13.3.1.3.2 Consolidation grouting shall always be done after the backfill grouting is completed in a length of at least 60 m ahead of the point of consolidation grouting.

13.3.1.3.3 Consolidation grouting should be usually done all round the bore, and for a uniform radial distance from the finished concrete face. The extent of grouted rock mass should be determined by the designer based on the design of the concrete lining and the extent to which cracks are assumed to extend into rock when the lining is stressed by internal pressure. Usually, the depth should be between 0.75D to D where D is the finished diameter of the tunnel, except in special reaches where it could be more.

13.3.1.3.4 The pattern of grout holes for consolidation may be a set of holes in one vertical plane, such a plane being called the grout plane. The spacing of the grout planes will depend upon the structural formation of rock and the travel of grout at the specified pressure. The exact spacing as in the case of contact grouting should also be adjusted in the field to suit the length of the shutter used for concreting. In this plane the number of holes may normally be 4 for small size tunnels and 6 for large size tunnels. The arrangement should be staggered in alternate grout planes, by about half the spacing between the holes along the periphery in the plane. In special locations the number of holes may be increased. The top three holes in grout pattern may be used for both backfill, contact and consolidation grouting.

13.3.1.3.5 Around shafts and large opening like powerhouse, the grout pattern will be similar, but the number of holes in the plane may be increased depending on the size, but the spacing should generally exceed the depth of the hole.

13.3.1.3.6 Contact grouting would not generally be necessary where consolidation grouting is to be done. However, it should be decided by actual contact grouting in jump holes after consolidation grouting.

13.3.1.3.7 Depending upon the rock formations and the grout intake, the consolidation grouting should be done in one or more stages with increasing pressures.

13.3.1.3.8 Maximum grout pressure should not normally exceed twice the design load on lining or supporting systems as the case may be.

13.3.1.4 *Permeation grouting*

13.3.1.4.1 The aim of pre-excavation permeation grouting is to reduce water ingress and hydraulic conductivity of rock mass surrounding tunnel wall by permeation of suitable grout under pressure through rock mass/fractures and creating a grout envelope. Such type of grouting is usually done ahead of tunnel face before tunnel excavation.

13.3.1.4.2 The direction and length of grout holes is fixed based on orientation and location of major rock fractures to cross maximum permeable features. Number of grout holes and grout take usually depend on the grouting stage, tunnel size and targeted ingress limit.

13.3.1.4.3 When tunnel strata permit, length of the grout holes should be kept 15 m to 20 m ahead of tunnel face, so that atleast 5 m thick grouted envelope is available even after three to four rounds of excavation. The overlap distance between subsequent grout fans shall be 5 m or more.

13.3.1.4.4 Lookout radial distance between end of each hole and to be excavated tunnel profile should be kept more than adopted rock bolt length.

13.3.1.4.5 The grouting material should depend on aperture of features, quantum of ingress and targeted ingress value. For smaller openings where Ordinary Portland cement does not permeate, micro-fine cement [*see* IS 16993] should be used in grout mix. Sand is not used for permeation grouting.

13.3.1.4.6 Grouting should be done with stable grout with suitable bleeding, viscosity, strength and setting time [*see* IS 16993]. Cement should be mixed in a high shear vane mixer of more than 1 400 RPM, with varying water cement ratio and superplasticizer depending on desired grout properties and grout acceptability by rockmass/fractures to be permeated.

13.3.1.4.7 The grouting pressure should be sufficiently higher than the hydrostatic pressure to overcome the water ingress. Grouting should be started from holes near invert and gradually proceed towards side walls and then to crown.

13.3.1.4.8 Suitable stop criterion for deciding grouting of holes should be kept depending on project restriction for allowable water ingress or volume of grout take or achievement of required grouting pressure.

13.4 Pressure to be Used for Grouting

13.4.1 The pressures to be used for grouting will depend on the rock characteristics, the design requirements and the rock cover. With adequate rock cover (more than 3 times the diameter of the tunnel), the other two will govern. For backfill grouting, the maximum recommended pressure is 0.5 N/mm^2. For consolidation grouting, a maximum pressure of 0.7 N/mm^2 is normally recommended but this may be increased up to 2.0 N/mm^2 in special cases provided that there is adequate cover and the joints in the rock are not likely to open up by this pressure. This pressure can be applied from inside the rock and not close to the concrete lining.

13.4.2 The pressure gauge should be watched constantly so that the pressure on the grout is regulated as long as grouting is in progress. Any desired increase or decrease in the grouting pressure is obtained by changing the speed of the grout pump. When the grout in the supply line becomes slugging, the grout-hole valve should be closed and the blow off valve opened so that the supply line may flush or washed.

13.5 Testing for Efficacy of Grouting

This testing may be done by drilling the holes in between the grout planes and by testing water intake in these test holes. If this is compared with the water test made before the grouting, this water will give an indication of the efficacy of grouting. Further grouting of this test hole and intake in this hole will give further indications. It is only after these tests that the engineer-in-charge may decide on increasing the number of grout planes, if required.

13.6 Capacity of Grouted Arch

The capacity of grouted arch is given as:

$$P_{gt} = \frac{2q_{gt}l_{gt}}{BF_{gt}} \qquad \ldots (13)$$

where

q_{gt} = uniaxial compressive strength of grouted rock mass (in t/m^2),
l_{gt} = thickness of grouted arch (in m),
B = size of opening (in m),
F_{gt} = mobilization factor of grouted arch
= $95\ P_{roof}^{-0.35}$, and
P_{roof} = ultimate support pressure in roof of tunnel (in t/m^2).

13.7 In case of water-charged rock mass or post construction saturation of rock mass, heavy support pressure and seepage pressure may develop on the concrete lining or shotcrete lining. The extra high support pressures/seepage pressures may be taken care off by grouted arch.

14 DESIGN OF INTEGRATED SUPPORT SYSTEM

14.1 General

14.1.1 A semi-empirical approach is used to determine the capacity of support system consisting of shotcrete, reinforced rock arch, steel rib and grouted arch. In Fig. 30, the dotted line shows the effective width, l_1 of the reinforced rock arch. The load carrying capacity of the reinforced rock arch is dependent on the minimum uniaxial compressive strength of the reinforced rock arch.

14.1.2 Figure 30 shows that the total support pressure ($P_{roof} + U$) will be equal to the sum of capacities of shotcrete, reinforced rock arch, steel rib and grouted arch. Simple hoop action is assumed as illustrated in Fig. 30. The arch subtends an angle of 2θ at its centre which is about 90° for tunnels. Ultimate support pressure = Total capacity of support system, which is given by

$$\left(u + P_{roof}\right) = P_{sc} + P_{bolt} + P_{gt} + P_{rib}$$
$$= \frac{2q_{sc}t_{sc}}{F_{sc}B} + \frac{2q_{crm}l}{F_{s}B} + \frac{2q_{gt}l_{gt}}{F_{gt}B} + \frac{P_{rib}}{S_{rib}B} \qquad \ldots (14)$$

where

P_{roof} = support pressure and is the seepage water pressure.

NOTE — In the above equation, it is assumed that shotcrete will shear along a length of arch approximately equal to ($F_{sc}\ B$) , where all the notations in above equation have the same meaning as described earlier.

14.1.3 The following trends have been obtained:

a) Pre-tensioned bolt is more effective in good rock conditions. The efficiency of pre-tensioned bolt decreased slightly for poor rocks due to creep and loss of tension.
b) The full-column-grouted-untensioned anchors are more effective in poor rock conditions than in good rock conditions. The reason may be that the anchors are subjected to large radial strains in poor rock masses leading to more tension induced in anchors. Figure 31 illustrates better performance of grouted anchors.

14.2 Application of Semi-Empirical Design Approach

14.2.1 For tunnels located near faults/thrusts (with plastic gauge) in seismic areas, the ultimate support pressures may be increased by about 25 percent to account for accumulated strain in the rock mass along the fault. If the tunnel is away from the fault by 2*B*, the seismic effect is negligible.

14.2.2 The support pressure, due to squeezing out of gouge from the shear zone, may be estimated by applying Terzaghi's theory of arching which indicates that the support pressure, with gouge, will increase the width of the shear zone. As such, the treatment of shear zone is essential to bear the high support pressure as shown in Fig. 5. However, in this shear zone (say < 250 mm), the support pressure is very small and hence, there is no need for special shear zone treatment (*see* **8.1**).

14.2.3 The capacity of shotcrete and reinforced rock arch is calculated by trial and error. The design parameters are selected, so that ultimate pressure is equal to the design capacity. If support pressure is high, say more than 0.5 N/mm^2, steel ribs may be used and embedded in shotcrete. The spacing of steel ribs may be estimated until equation (12) is satisfied. The design philosophy is illustrated in Fig. 32. In case of water charged rock mass, experience shows that design tables give useful parameters as they neglect the seepage pressure. Equation (14) may then be used to find out the extent up to which rock mass should be grouted. Grouting is possible generally where thick shotcrete has been provided to take high grouting pressure.

14.2.4 In very poor rock conditions, assumptions are generally invalid. Hence, special specifications need to be followed to treat thick shear zones, rock burst conditions and highly squeezing conditions.

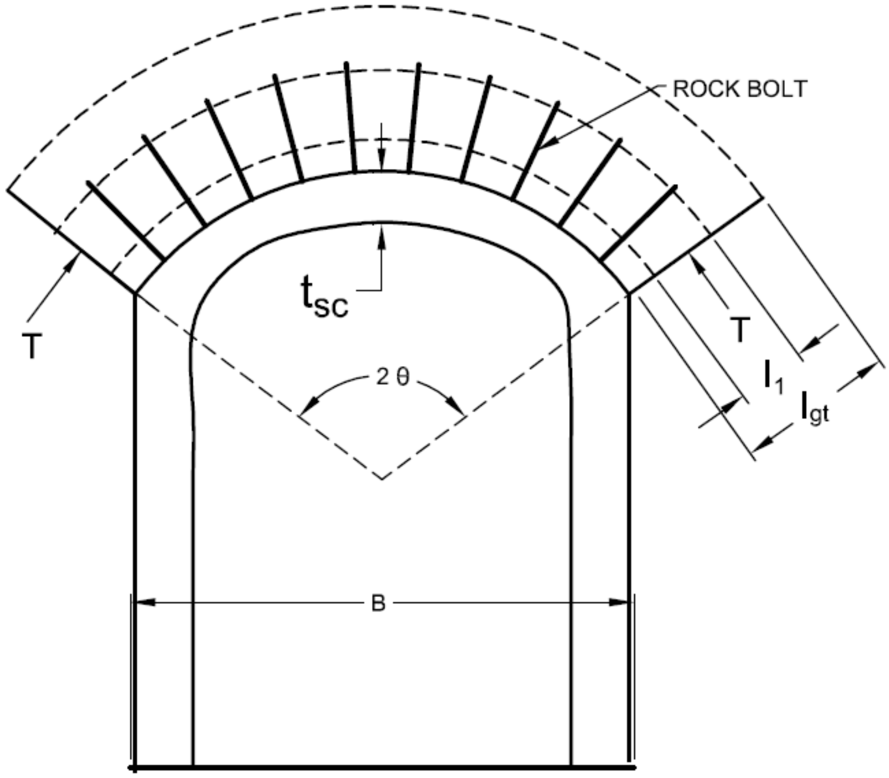


Fig. 30 Capacity of Reinforced Rock Arch

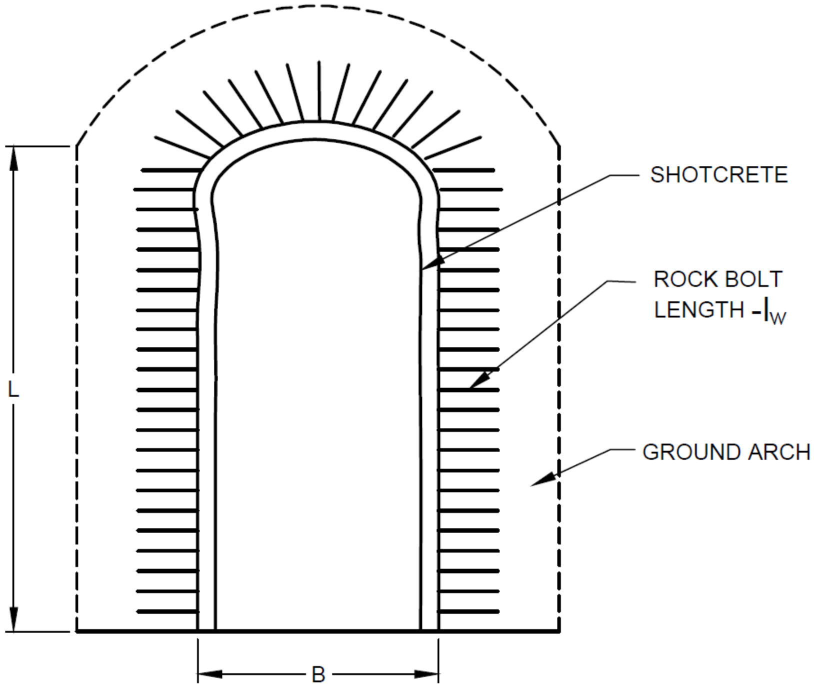
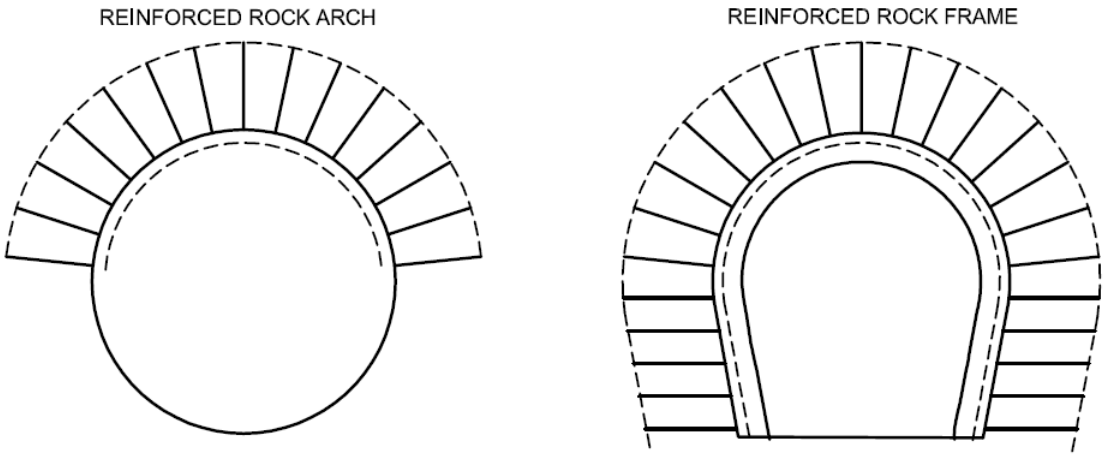
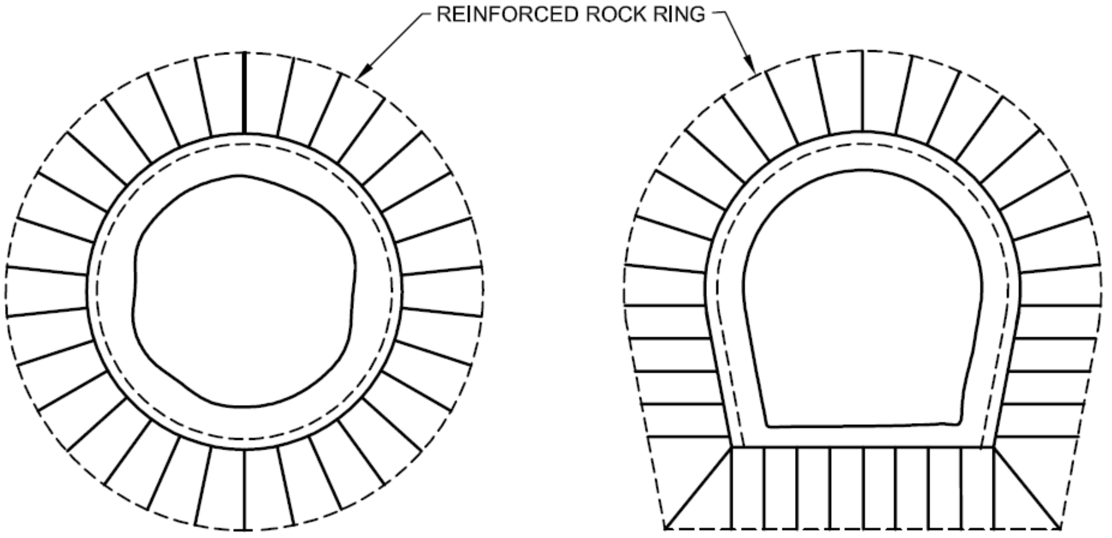


Fig. 31 Design of Wall Reinforcement of Cavern



32A Non-Squeezing Ground

32B Mild Squeezing Ground



32C Moderately Squeezing Ground

32D Moderately Squeezing Ground

Fig. 32 Design Philosophy of Rock Reinforcement in Tunneling

14.2.5 In case of water and power tunnels, seepage pressure may be assumed equal to the internal water pressure and the worst case is when the tunnel is empty and seepage water pressure acts on the shotcrete lining. If required, rock mass may be grouted to take high support and seepage pressure, alternatively, concrete lining may be designed according to the design criteria.

14.3 Tunnel Through Intra Thrust Zone

In the Himalayas, the tunnels have to pass through intra thrust zone in some complex geological and tectonic situations. The faults and thrusts are subjected to slip over a long period of time due to very slow tectonic movement of the Indian Plate with respect to the Chinese Plate. It is therefore essential to build a segmented concrete lining so that segments of the concrete lining can slip with respect to each other with time. The design decision should be taken on the basis of instrumentation plate within the intra thrust zone.

14.4 Experience in Poor Rock Conditions

14.4.1 Fibre reinforced shotcrete is very successful in weak rock masses. The main advantage is that lesser thickness of fibre reinforced shotcrete is needed. No welded mesh is required to reinforce shotcrete. Its rebound is less due to fibres, provided shotcrete is graded properly and sprayed properly.

14.4.2 It is recommended that the mesh should not be raised where even surface of tunnel is not available due to over break, provided shotcreting is used. However, thickness of shotcrete should be increased by 10 mm.

15 SPECIAL REQUIREMENTS

15.1 For treatment of shear zone, crossed rock anchors/bolts should be provided across shear zones. Further, the gouge should be cleaned to the desired extent, anchors are provided and connected to welded mesh. Final, dental shotcrete is back filled. In wide shear zone, reinforcement in shotcrete is also placed to withstand high support pressure. Anchors should be inclined according to the dip of shear zone to stop squeezing of gouge and thereby stabilize the deformations.

15.2 In case of steel ribs in large tunnels and caverns, haunches should be strengthened by additional anchors to withstand heavy thrust due to the ribs.

15.3 In case of poor rock mass, spiling bolts (inclined towards tunnels face) should be installed before blasting to increase the stand-up time of tunnel. Roof shotcrete is then sprayed on roof. Then spiling bolts are installed. In the final cycle, roof bolts are installed.

15.4 In case of argillaceous rocks and swelling rocks where its bond with shotcrete is poor, thickness of shotcrete may be increased by about 30 percent.

15.5 In reaches of very poor rock masses, steel ribs should be installed and embedded in shotcrete to withstand high support pressures.

15.6 In rock burst prone reaches, resin anchors and fibre reinforced shotcrete should be used to increase ductility of support system and to convert brittle mode of failure into the ductile mode of failure. Stress release holes and stress relief blasting can also be used to release/minimize stresses.

15.7 In highly squeezing ground condition ($H > 350\ Q^{1/3}$ m and J_r/J_a, < 1/2, where J_r is joint roughness number and J_a is joint alteration number [*see* IS 13365 (Part 2)] steel ribs with struts should be used when shotcrete fails repeatedly in spite of more layers. With steel ribs, excavation by force-poling is easily done by pushing steel rods into the tunnel face and welding other ends to ribs. Floor heaving can be prevented by rock bolting of floor. Some delay, but less stand-up time is necessary to release the strain energy of the broken zone. Smaller blast holes (approximately 1 m) will also be of help. Instrumentation of broken zone is needed. It should be ensured that the tunnel closure is arrested before it reaches the 5 percent of the width of the tunnel.

15.8 In case of unstable portal, horizontal anchors of equal length should be provided inside the cut slope, so that it acts as a reinforced rock breast wall.

15.9 In case of deep and long tunnels in complex geological conditions, probe hole of 20 m length should be drilled inside the tunnel face to get accurate picture of geological conditions in advance of tunnelling. The probe hole will also dissipate seepage pressure slowly in the water charged rock mass which is likely to be punctured during tunnelling. This will also avoid flash floods soon after blasting and consequent loss of life and support system.

15.10 Concrete lining for water/pressure tunnels should be laid far away from the tunnel face where the broken zone is stabilized, that is about four times the radius of broken zone. The concrete lining should be segmented within active thrust zone to allow relative movement along the faults/thrust.

15.11 Grouting of long bolts is sometimes not done satisfactorily due to the difficulty of supervision and expanding agent (aluminium powder) is seldom added. So pull out tests should be conducted on at least 2 percent bolts to check the quality of bolts. If required extra bolting should be done to strengthen the support system.

ANNEX A

(*Foreword*)

COMMITTEE COMPOSITION

Rock Mechanics Sectional Committee, CED 48

Organization	*Representative(s)*
Indian Institute of Technology Roorkee, Roorkee	Dr N. K. Samadhiya (***Chairman***)
AIMIL Ltd, New Delhi	Shri Akhil Raj
Amberg Engineering AG India Private Limited, Gurugram	Shri Kripal Choudhary Shri Rakesh Pandita (*Alternate*)
Border Roads Organisation, New Delhi	Lt Col Anil Raj
Central Board of Irrigation and Power, New Delhi	Shri G. P. Patel Shri Uday Chander (*Alternate*)
Central Soil and Materials Research Station, New Delhi	Shri Hari Dev Shri Mahabir Dixit (*Alternate*)
Central Water and Power Research Station, Pune	Shri Rizwan Ali Dr S. A. Burele (*Alternate*)
Central Water Commission, New Delhi	Member (D & R) Chief Engineer, Design (NW & S) (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	Dr Shantanu Sarkar Shri Koushik Pandit (*Alternate*)
CSIR-Central Institute of Mining and Fuel Research, Dhanbad	Dr J. K. Mohnot Dr R. D. Dwivedi (*Alternate*)
CSIR-Central Road Research Institute, New Delhi	Dr Pankaj Gupta Shri R. K. Panigrahi (*Alternate*)
Engineers India Ltd, New Delhi	Dr Altaf Usmani Shri Saikat Pal (*Alternate*)
Geological Survey of India, Kolkata	Shri Santosh Kumar Tripathi Shri D. P. Dangwal (*Alternate*)
Indian Institute of Technology Delhi, New Delhi	Prof R. Ayothiraman Prof Prashanth Vangla (*Alternate*)
Indian Institute of Technology Roorkee, Roorkee	Dr Mahendra Singh Dr Priti Maheshwari (*Alternate*) Prof Sumit Sen (*Young Professional*)
Indian Institute of Technology-ISM, Dhanbad	Prof A. K. Mishra Dr R. K. Sinha (*Alternate*)
Indian Society for Rock Mechanics and Tunnelling, New Delhi	Dr C. S. Khokhar
Irrigation Research Institute, Roorkee	Shri Dinesh Chandra Shri Shankar Kumar Saha (*Alternate*)
National Disaster Management Authority, New Delhi	J S (Mitigation) Dr Ravinder Singh (*Alternate*)
National Highways and Infrastructure Development Corporation Limited, New Delhi	Shri Ashok Kumar Jha
National Highways Authority of India, New Delhi	Representative

Organization	*Representative(s)*
National Hydroelectric Power Corporation Ltd, Faridabad	Shri Rajesh Kumar Shri Ajay Kumar Verma (*Alternate* I) Shri Pradeep Kumar Garnayak (*Alternate* II)
National Institute of Rock Mechanics, Bengaluru	Dr A. Rajan Babu Shri G. Gopinath (*Alternate*)
NTPC Ltd, New Delhi	Shri N. K. Jain Shri Prakash Rawat (*Alternate*)
Rail Vikas Nigam Ltd, New Delhi	Shri Sumit Jain Shri Vijay Dangwal (*Alternate*)
Reliance Industries Limited, Mumbai	Shri Abhishek Sampatrao Bhosale
RITES Ltd, Gurugram	Shrimati Jyotsna Dixit Shri Sandeep Singh Nirmal (*Alternate*) Shri Ahmed Shaz (*Young Professional*)
THDC India Limited, Rishikesh	Shri T. S. Routela Shri Samrat Mukherjee (*Alternate*)
In Personal Capacity,(*Jal Vidyut Apts, Sector 21-C, Faridabad - 121001, Haryana*)	Shri Imran Sayeed
In Personal Capacity, (*C-1004, Park View City-2, Sector-49, Gurugram*)	Dr Manoj Verman
In Personal Capacity, (*Flat No. 4123, Tower 4, ACE Golfshire, Sector-150, Noida*)	Shri R. K. Goel
In Personal Capacity, (*A-125. Prodhyagiki Apartment, Sector-3, Dwarka, New Delhi*)	Dr Rajbal Singh
In Personal Capacity, (*Naimex House, A-8, Mohan Co-operative Industrial Estate, Mathura Road, New Delhi - 110044*)	Shri Sharique Khan
BIS Directorate General	Shri Arunkumar S., Scientist 'E' and Head (Civil Engineering) [Representing Director General (*Ex-Officio*)]

Member Secretary

Dr Manoj Kumar Rajak
Scientist 'D' (Civil Engineering), BIS

Bureau of Indian Standards

BIS is a statutory institution established under the *Bureau of Indian Standards Act*, 2016 to promote harmonious development of the activities of standardization, marking and quality certification of goods and attending to connected matters in the country.

Copyright

BIS has the copyright of all its publications. No part of these publications may be reproduced in any form without the prior permission in writing of BIS. This does not preclude the free use, in the course of implementing the standard, of necessary details, such as symbols and sizes, type or grade designations. Enquiries relating to copyright be addressed to the Head (Publication & Sales), BIS.

Review of Indian Standards

Amendments are issued to standards as the need arises on the basis of comments. Standards are also reviewed periodically; a standard along with amendments is reaffirmed when such review indicates that no changes are needed; if the review indicates that changes are needed, it is taken up for revision. Users of Indian Standards should ascertain that they are in possession of the latest amendments or edition by referring to the website-www.bis.gov.in or www.standardsbis.in.

This Indian Standard has been developed from Doc No.: CED 48 (17934).

Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

BUREAU OF INDIAN STANDARDS

Headquarters:

Manak Bhavan, 9 Bahadur Shah Zafar Marg, New Delhi 110002
Telephones: 2323 0131, 2323 3375, 2323 9402 *Website*: www.bis.gov.in

Regional Offices: *Telephones*

Central : 601/A, Konnectus Tower-1, 6th Floor, DMRC Building,
Bhavbhuti Marg, New Delhi 110002 2323 7617

Eastern : 8th Floor, Plot No 7/7 & 7/8, CP Block, Sector V,
Salt Lake, Kolkata, West Bengal 700091 2367 0012, 2320 9474

Northern : Plot No. 4-A, Sector 27-B, Madhya Marg
Chandigarh 160019 265 9930

Southern : C.I.T. Campus, IV Cross Road, Taramani, Chennai 600113 2254 1442, 2254 1216

Western : Plot No. E-9, Road No.-8, MIDC, Andheri (East),
Mumbai 400093 2821 8093

Branches : AHMEDABAD. BENGALURU. BHOPAL. BHUBANESHWAR. CHANDIGARH. CHENNAI. COIMBATORE. DEHRADUN. DELHI. FARIDABAD. GHAZIABAD. GUWAHATI. HIMACHAL PRADESH. HUBLI. HYDERABAD. JAIPUR. JAMMU & KASHMIR. JAMSHEDPUR. KOCHI. KOLKATA. LUCKNOW. MADURAI. MUMBAI. NAGPUR. NOIDA. PANIPAT. PATNA. PUNE. RAIPUR. RAJKOT. SURAT. VISAKHAPATNAM.

Published by BIS, New Delhi